



TEXAS HIGHWAY DEPARTMENT

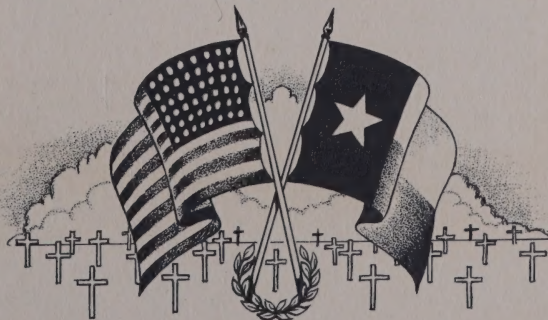
Construction & Maintenance

BULLETIN No. 11

1952



The Memorial Student Center
TEXAS A+M COLLEGE



HIGHWAY SHORT COURSE ISSUE

Foreword

This bulletin is published for the purpose of presenting practical information applicable to current highway problems. These conclusions and data are not necessarily endorsed by the Highway Department nor are they to be construed as instructions. Employees are invited and urged to submit suggestions and relate experiences which would benefit those engaged in the construction and maintenance of highways.

Photographic services are available and other assistance may be obtained if desired in preparing material for submission. All material and comments should be directed to the Chief Engineer, Construction & Maintenance, Austin, Texas.

Published Jointly by

Maintenance Division.....	D- 3
Equipment & Procurement Division.....	D- 4
Construction Division.....	D- 6
Traffic Services Division.....	D-16

COMPILED AND EDITED
INFORMATION SECTION,
TRAFFIC SERVICES DIVISION
REPRODUCED
OPERATIONS DIVISION
TEXAS HIGHWAY DEPARTMENT
AUSTIN 14, TEXAS

Equipment & Procurement

Construction

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Maintenance

Traffic Services



GREETINGS . . .

The Twenty-Sixth Annual Highway Short Course held at the Memorial Student Center, Texas A & M College, dealt with practically every phase of highway engineering and procedure. Attendance of highway employees at this Short Course was the largest in history. We were honored to have several retired highway personnel attend this meeting, lending their prestige and permitting us to continue benefiting from their wisdom and experience, gained throughout the years with the organization.

The program was well planned and the material was presented in an interesting and educational manner. Special appreciation should be extended to the Program Committee for their fine work: Messrs. Frank Maddox, Chairman, J. C. Dingwall, P. S. Bailey, and Fred J. Benson of A & M College.

The papers and discussions presented at the Short Course, along with photographs, are reproduced herein for the benefit of those who were unable to attend, and for general reference purposes.

D. C. Greer
State Highway Engineer

Twenty-Sixth
Annual Short Course
in
HIGHWAY ENGINEERING

TO BE HELD AT
COLLEGE STATION, TEXAS

UNDER THE DIRECTION OF THE

School of Engineering

DEPARTMENT OF

Civil Engineering

OF

The A. & M. College of Texas

AND IN COOPERATION WITH THE

State Highway Department of Texas



MARCH 5 AND 6

1952

GENERAL INFORMATION

Purpose.—The purpose of the course offered by the Texas A. & M. College is to bring together the engineers, contractors, officials, and others who are interested in the street and highway problems of Texas, in order that they may not only hear various phases of these problems discussed by experts of state and national reputation, but also that they may have an opportunity to meet with each other and exchange ideas.

The Course.—The Short Course will cover a period of two days, the program beginning on Wednesday, March 5, 1952 and ending Thursday, March 6, 1952 at 12:00 noon.

Fees.—There will be a registration fee of two dollars.

Discussion.—Discussion is essential to the success of the course and those in attendance are urged to enter discussions as far as time will permit.

Quarters.—Registrants will be housed at hotels in Bryan (5 miles north of College Station) at the Aggieland Inn on the College Campus, and in College dormitories. Several tourist courts are also available in Bryan and at College Station.

Meals may be had either in Bryan or at College.

Reservations.—Requests for reservations should be mailed directly to F. J. Benson, College Station, Texas or to John Blair, District Engineer, Texas Highway Department, Bryan, Texas.

Program.—The program for the Twenty-sixth Annual Highway Short Course was prepared by the Texas Highway Department Short Course Committee:

Mr. Frank S. Maddox, Chairman
Assistant State Highway Engineer

Mr. J. C. Dingwall
Engineer of Road Design

Mr. P. S. Bailey
State Maintenance Engineer

and for Texas A. & M. College:

Fred J. Benson
Professor of Civil Engineering

Administration.—The Highway Short Course is one of the yearly activities of the Civil Engineering Department and is given in cooperation with the Texas Highway Department. F. J. Benson, Professor of Civil Engineering is the Director of the Highway Short Course.

A business meeting of the District Engineers and other Texas Highway Department Officials will be held on Tuesday, March 4, in the Ballroom of the Memorial Student Center. Registration for the Short Course for those attending this meeting will be held in the Lounge of the Memorial Student Center from 8:00 a.m. to 10:00 a.m.

PROGRAM

All Meetings in Ballroom, Memorial Student Center

WEDNESDAY, MARCH 5, 1952

Morning

8:00 Registration, Memorial Student Center



Chairman: Dr. S.R. Wright, Head
Civil Engineering Department
Texas A. & M. College
College Station, Texas



10:30 Invocation
Rev. James F. Jackson
First Methodist Church
College Station, Texas

Address of Welcome
Gibb Gilchrist, Chancellor
Texas A. & M. College System
College Station, Texas



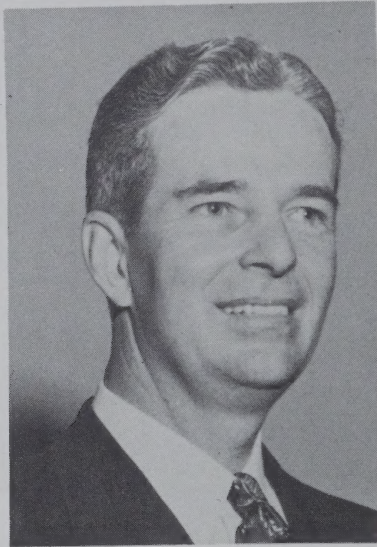
10:45 Response
Ed Bluestein, District Engineer
Texas Highway Department
Atlanta, Texas



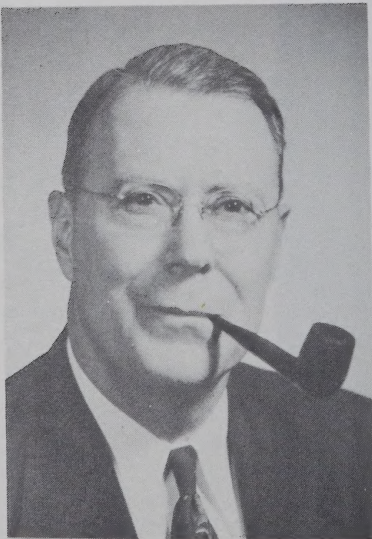
11:00 Selected Subject
Honorable E. H. Thornton, Jr., Chairman
Texas Highway Commission
Galveston, Texas

Importance of Public Relations
Honorable Fred A. Wemple, Member
Texas Highway Commission
Midland, Texas

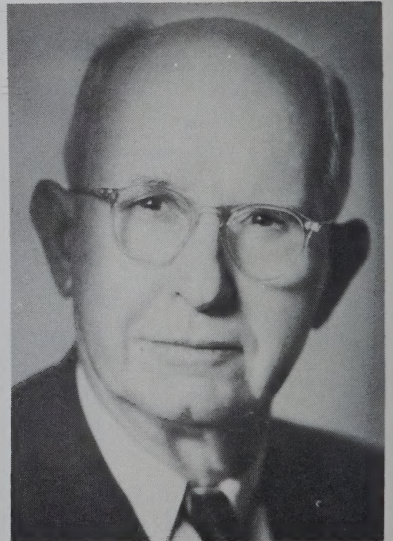
Selected Subject
Honorable Robt. J. Potts, Member
Texas Highway Commission
Harlingen, Texas



Mr. Thornton*

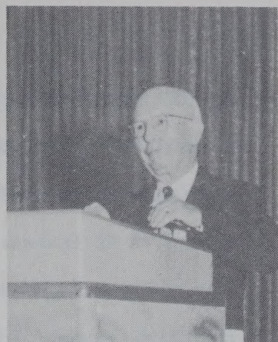


Mr. Wemple



Mr. Potts

*Because of an unexpected emergency, Mr. Thornton was unable to attend the Short Course.



Robt. J. Potts
College Station
3-5-52

TESTING OF HIGHWAYS BY CONTROLLED TRAFFIC

In beginning this discussion of full scale, accelerated traffic tests of highways, it may be well to point out that such experiments are of value in two distinctly different ways; first, they can assist in the proper determination of load limits and perhaps of load-zoning requirements; and second, they can be made to furnish additional data for the design of future construction.

In the United States one such test has recently been completed, construction of one other is starting this spring, and two others have been proposed. The famous Bates test of the early twenties, though conducted under vastly different traffic conditions, furnishes some fundamental data which are still of use in the design of rigid pavements.

Some general information regarding the modern road tests may be of interest.

Road Test One-MD, the best known of the modern series, was conducted by the Highway Research Board on behalf of eleven states, the District of Columbia, and the Bureau of Public Roads, as well as members of the truck manufacturing and the oil industries. The one mile test section was a part of U.S. Highway 301, near La Platte, Maryland. Test traffic on this nine-year-old twenty-four foot wide, 9-7-9 inch, rigid pavement was started in June, 1950. The test trucks operated 24 hours per day, 7 days a week until late in December 1950.

Four different axle loadings, each applied to a separate single lane section of the road, were used in this test. These were 18,000 pounds and 22,400 pounds on single axles, and 32,000 pounds and 44,800 pounds on tandem axles. Measurements were made of crack damage, settlement of slabs, pumping, roughness, stresses in the slabs induced by various loads, strength of the concrete, the quality of the subgrade, etc. Cost was estimated at \$245,000.

The next such project to be proposed was the WASHO Test Road, to be constructed along US Route 191, near Malad, Idaho. The Highway Research Board will

conduct this experiment also. Sponsors are ten Western states, including Texas, as well as the Bureau of Public Roads and certain industries. The test section, now under construction, will be of the flexible type, with pavement depths varying from 6 to 22 inches. Two identical loops, each with two 1900 ft. two-lane straight-a-ways, will be constructed this spring. Test traffic will start next summer, and the first report of results will be available at the time of the winter shut-down. Traffic will be resumed in the spring of 1953 and will continue for about three months. Trucks are to operate twenty hours daily, six days per week, for a total period of twelve months.

Single axle trucks will run on one loop and tandem axle trucks on the other. Load magnitudes will be kept within the limits of those observed in normal highway traffic. Preliminary testing of the proposed materials and sub-grade soils have been completed by several of the participating states, including Texas. During the traffic test, measurements of structural damage, roughness and other variables will be made at frequent intervals on all sections. Cost is estimated at \$275,000.

Another test proposed, but, according to my information, not yet definitely adopted by the states concerned, is the so-called "Comprehensive Test" of the Mid-West. Principal sponsors are to be the states represented by the Mississippi Valley ASHO, though others may be included. This ambitious undertaking, planned by the Highway Research Board, besides evaluating the effect of some eight different load types (four single axle and four tandem axle loads), will also provide data for comparing the performance of flexible pavement with rigid pavement under identical conditions of subgrade, climate and traffic. Four oval loops are planned, each loop to have rigid pavement on one of the 6600 ft. two-lane tangents and flexible pavement on the other. Various thicknesses of both types are to be tested. Trucks will operate 20 hours daily, six days per week for twelve months. All possible measurements of the variables involved will be made. The cost is roughly estimated at \$1,000,000.

• A fourth test, to be sponsored by states forming the South Eastern ASHO is under consideration, but plans are still incomplete.

The preliminary results of the Maryland test have already been rather widely circulated and need not be here repeated. Furthermore, it is hazardous, perhaps

presumptious, to draw any general conclusions before the final reports of the Highway Research Board, with complete analyses and finding, are in hand. But for the benefit of a later part of this paper I shall advance a few very tentative suggestions:

1. At about 92,000 truck-load passes, just before the heaviest units were withdrawn from the test, it seems to appear, by crack measurements only, that:
 - (a) The 44,800 pound tandem job had done 11 times as much damage as the 32,000 pound tandem, 18 times as much damage as the 22,400 pound single axle, and 93 times as much damage as the 18,000 pound single axle.
 - (b) The 32,000 pound tandem had done $1 \frac{2}{3}$ as much damage as the 22,400 pound single axle and $8 \frac{1}{4}$ times as much as the 18,000 pound single axle.
 - (c) The 22,400 pound single axle had done $5 \frac{1}{2}$ times as much damage as the 18,000 pound single axle.
2. From all the data, particularly the strain gauge readings, it appears that, if two axles are spaced at the usual distance for tandem axles, about 44 to 52 inches, the area of influence, on rigid pavements, of one axle overlaps the other area and the ensemble must be regarded as a unit -- neither as a single axle nor as two separate single axles.

From the Maryland results it has been possible to make some interesting comparisons of the probable effect on highway life of post-war increases in traffic. From the observed effect of the four loads used on the four sections, it is possible to estimate what the damage would have been had any given combination of these or other loads been applied in a repetitive pattern to a single section. For instance, it has been estimated that the life expectancy of this Maryland test pavement would have been about 26 years if continuously subjected to the type of traffic which moved over US 90 between Houston and Beaumont during the year 1947. If the Maryland pavement carried our US 90 traffic of 1948, it would have endured 14 years. Under 1950 traffic its estimated life would have been only 6 1/2 years. But if every overloaded axle in this 1950 traffic had first been reduced to the Texas legal load limit, then the life of the pavement would have been eleven years, an increase of 4 1/2 years.

Just as an indicator that more actual experimental data as a basis of road

design is really needed, it is interesting to note that the Idaho test, even in the preliminary stage, has furnished some significant differences in the viewpoints of the participating states. For instance, the depths of pavement recommended by four states varied from 8 inches to 17 inches, though all four presumably tested identical samples of materials and designed for the same axle load. (Incidentally, Texas submitted the low figure). May we not hope that such disagreements will be resolved by the indisputable results of full-scale testing under traffic, and will lead to progress in the development of better methods of design by all concerned?

Now that my paper is nearing its close, you will no doubt be surprised when I say that all that has preceded was intended as scenery, background, or atmosphere for your consideration of a question which was, in the first place, the real reason for this paper. You engineers may not get to do much of the voting, but you do most of the thinking and planning. As in many other matters, a consensus of your opinions would be most welcome in this case.

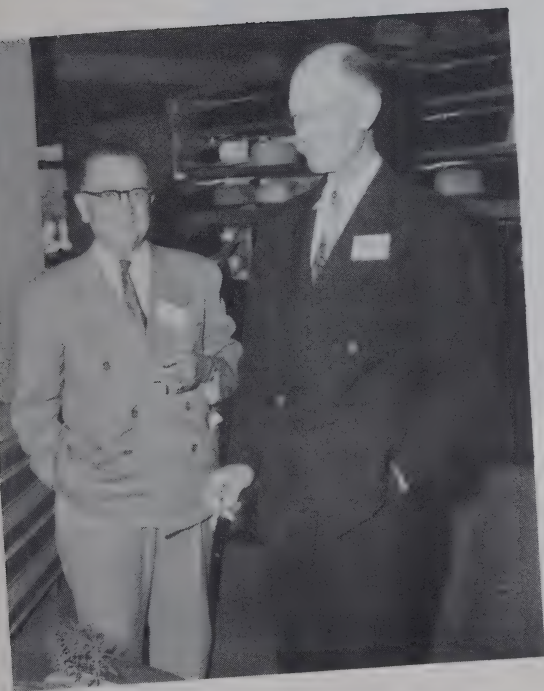
So, to each of you, I pose this question: Should we here in Texas conduct a road test, or tests, of our own in an attempt to solve more promptly at least some of the design problems facing us, and to provide more definite information on permissible load limits? Might not some rather brief, limited, and not-too-technical tests, made on short sections of existing highways quickly furnish some of the data needed for design, and for the proper load-zoning of our land service roads, and also assist in determining more accurately the desirable load limits on some of our existing highways?

An elaborate, complete, technical project, such as the three tests above described, seems to be out of our reach financially, and also because it would create a serious drain on our trained, technical personnel at a time when the work load is already high. If, however, we rely completely for our factual field information on tests conducted in distant regions, perhaps under climatic and soil conditions different from our own, is there not a possibility that the results may not always fit our exact needs? Again, tests made within the state, conveniently accessible for inspection as well as reports, would carry a much more convincing appeal to all of us than one far away even though it be more complete and fully reported. Still another consideration is the time element. It is nearing two

years since the trucks started rolling on the Maryland test; fourteen months since they stopped. But the final report, with conclusions, is not yet available. There are indications that the Idaho test, in which we are participating, will mature more rapidly, but final results can hardly be expected in less than eighteen months or two years. In the meantime, just "how bad are we hurting" anyway?

My own mind is open. I make no recommendation. May I remind you of the old saying: "In multitude of counsel there is great wisdom". I know of no group better prepared than are you to "kick this idea around", and come up with some helpful analyses and suggestions. Let us hear from you.

• • •



Faces around the Short Course . . .

Afternoon

Chairman: George M. Garrett, Engineer-Manager
Fort Worth Urban Project
Texas Highway Department
Fort Worth, Texas



2:00 National Conditions and the Texas Highway
Picture
D. C. Greer, State Highway Engineer
Texas Highway Department
Austin, Texas

EMERGENCY CONDITIONS AFFECTING
HIGHWAY CONSTRUCTION

2:20

The Steel Situation

Herbert Eldridge, Chief Engineer of Planning
Texas Highway Department
Austin, Texas



- THE STEEL SITUATION -

By

Herbert Eldridge, Chief Engineer of Planning,
Texas Highway Department

Presented to the 26th Annual Short Course in
Highway Engineering, College Station, Texas, March 5, 1952

One of the national weekly reports on business recently contained this item: "Some businesses are booming; others, even in the same industry, are depressed. Defense production is expanding; but many contracts and subcontracts are being cancelled because of the arms stretch-out. Many prices are firm, others are suffering anticipated price jitters. Some firms are screaming for metal; at the same time some steel producers are already running short shifts. One automobile manufacturer has no inventory to speak of; another's warehouse is full to bursting. And so on."

Considering the experience of the Texas Highway Department with the steel problem one might say that this pretty well summarizes the situation. Some steel suppliers claim to have plenty of steel; others are in short supply. Some reinforcing steel suppliers can make delivery on current quarter allocations; others require allocations from one to two quarters in advance. According to N.P.A. the steel fabricating shops need more work. To initiate more work for the shops we need additional steel allocations; N.P.A. does not come through with additional allocations. And so on.

Prior to the inauguration of the Controlled Materials Plan Mr. Greer decided that this phase of our activities would be handled by the Chief Engineers. All three major sections of the Headquarters Office, namely, Planning, Construction & Maintenance, and Operations, would be involved to some extent at least in the operation of the Controlled Materials Plan. It being the function of the Planning section, which includes the Divisions of Planning Survey, Materials & Tests, Aid Projects, Land Service

Roads, Road Design, and Bridge, to consider kind, quality, availability, and economics of construction materials in the design of projects, and to handle all preliminaries leading up to construction, it was agreed that the Planning section would handle the Controlled Materials Plan for projects, carrying on into the construction stage where necessary in the fulfillment of material availability. It was agreed that Mr. Waller, Chief Engineer of Construction & Maintenance, would handle the Controlled Materials Plan for all maintenance materials, maintenance equipment, and construction equipment.

In the Planning section the Designs Divisions of Land Service Roads, Road Design, and Bridge prepare the necessary applications for the initiation of projects that are their assigned responsibility for handling. In addition, the Bridge Division prepares all applications for allotments of the controlled materials of steel, copper, and aluminum to projects; prepares estimates and reports and maintains records. The Bridge Division also handles for the Chief Engineer of Planning the applications of other road and street agencies, such as the Counties and Cities.

After the Bureau of Public Roads had been designated as the claimant agency for controlled materials for all road and street work, and prior to the inauguration of the Controlled Materials Plan, Chief McDonald requested and President Anderson of the American Association of State Highway Officials appointed a committee of five from that organization to work with the Bureau in presenting steel requirements to N.P.A. and to assist in the preparation of data for distribution of steel allocations to States. Mr. Randle B. Alexander, our Bridge Engineer, is a member of that committee. His experiences and contacts have been of great value to the Department in directing our course of operation under the Controlled Materials Plan.

In the year 1900 the total annual steel production in the United States was 10 million ingot tons. Today the total annual steel production is approximately 108 million ingot tons. This gives an idea of the expansion of steel production in this country in the first half of this

century. With the completion of additional plant facilities now under construction, it is estimated that the 1954 production will approximate 120 million ingot tons. For the third quarter of 1951, the National Production Authority, acting on the assumption that roads and streets are expendable, allotted 300,000 tons of steel to the Bureau of Public Roads for use throughout the nation on the highway and street systems. For the first quarter 1952 the allotment was reduced to 250,000 tons. This furnishes to the highway and street systems less than 1% of the nation's steel production. The American Association of State Highway Officials, by resolution at its annual meeting in Omaha last October, requests a minimum of 2% for highway and street use.

For the fourth quarter 1951 Texas was allotted steel in the approximate amount of 20,000 tons, and for the first quarter 1952 about 15,000 tons. The second quarter 1952 allotment is 13,572 tons. The allotments are divided into three classes - structural shapes, plates, and other. The second quarter allocation on structural shapes was 4,118 tons, plates 930 tons, and other 8,524 tons. "Other" could be classed as "steel reinforcement" since this is its practical use.

Steel is subdivided into Class A and Class B products. All structural shapes, plates, reinforcing steel, and steel wire are Class A products. The steel allotments are for use in obtaining delivery of Class A products. Examples of Class B products are corrugated metal pipe and steel forms for concrete. Manufacturers of Class B products are granted steel allotments direct from the Industry Division of N.P.A.

The National Production Authority and the Bureau of Public Roads are to be commended for inaugurating procedures for prompt and efficient handling of the Controlled Materials Plan. N.P.A. has properly delegated much of its authority to the Bureau of Public Roads. The Bureau has in turn delegated authority to its Divisions and Districts to the extent that prompt action is assured.

To initiate a road or street project for use of controlled materials it is necessary that an application be forwarded through the State Highway Department, which establishes intent, essentiality of project, and other

data, including quantities of controlled materials. Upon approval the Bureau of Public Roads issues what is termed a letter of intent. This constitutes authority to proceed to the contract stage and to request assistance for delivery of controlled materials. After award of contract an application may be forwarded through the State Highway Department for allotment of controlled materials. Upon approval, the allotments are issued by the Bureau direct to the contractor.

The quarterly allotments of steel to Texas to date have been out of balance according to need. The quantity of structural shapes represents about 50% of the actual need. The quantity of plates is more than needed in some quarters and less than needed in other quarters. The quantity of "other" steel has exceeded the quarterly requirements to date.

In the use of steel allocations our first concern has been to service the projects under construction. This has been accomplished with very little delay in completion of projects. With an allocation of structural shapes of about half the quantity needed the Department has found it necessary to defer some projects involving substantial quantities of structural shapes. We have also made changes in designs to use more reinforced concrete and less structural steel. Through these measures we have succeeded in servicing all projects under construction, including Highway, City, and County work. The projects under construction will consume the allocations to Texas through the fourth quarter 1952. This assumes that the third and fourth quarter allotments will not be less than the amount received for the second quarter. Using the same assumptions, projects involving the use of structural shapes on which bids will be requested to be received in April will constitute charges against the first quarter 1953 allotment. In servicing construction projects we attempt to schedule deliveries by calendar quarters in such manner that deliveries will be made as the steel is needed. This works satisfactorily where mill acceptances are obtained. When mill acceptances are not obtained according to the allocations our entire plan is disrupted by revalidation to a subsequent quarter allocation and, in most cases, the loss of the original quarter allocation quantity of steel.

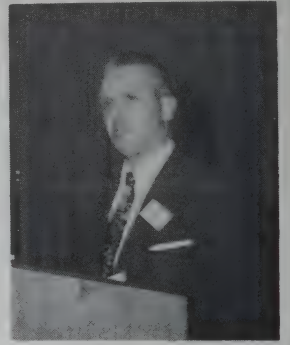
A plan is under consideration and may soon be approved by M.P.A. whereby projects involving the use of less than 25 tons of steel may be self certified. This plan would probably work similar to plan for industrial expansion whereby the contractor certified his steel orders according to the specified symbol and limitations imposed by the authority. Many of our field engineers desire to know what they can do to assist with the operation of the Controlled Materials Plan. The following is listed:

1. In the design of projects avoid the use of structural shapes and plates where practical.
2. Urge your contractors to use the steel allocations promptly when received, and to promptly return any unused allocation.
3. When information is requested of you regarding applications for controlled materials please refer such requests to the Austin Office. There are so many operational changes in the plan and procedures that it is impractical to keep the entire organization up to date.
4. At every opportunity, stress the importance of highways and highway transportation as essential to national defense and to the national economy. Bear in mind that the officials of D.P.A. and M.P.A. are not convinced of this essentiality.

We are most grateful to the highway contractors and steel suppliers for their indulgence and for their cooperation in making the plan work. While some projects have been tentatively deferred, others that do not involve the use of structural steel have moved to construction to the extent of available funds. While there may be a few "idle" projects, there are no "idle" funds. As long as available funds are going under the wheels of traffic on meritorious work the Department is doing its job. Steel may be in short supply but money for the needed highway improvements presents a more critical situation.

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2.40 National Scrap Drive and Its Application
to the Highway Department
Walter H. Leo, Supervisor
Steel Industry Scrap Mobilization Committee
Dallas, Texas



NATIONAL SCRAP DRIVE AND ITS APPLICATION
TO THE HIGHWAY DEPARTMENT

This object I have in my hand is commonly known as junk or scrap. It is the kind of thing you will find lying around farms, ranches, oil fields, all types of industry, and almost any place where articles of steel are used. In its present state, lying in those locations, it is absolutely of no value to anyone. As a matter of fact, it may be unsightly, or even hazardous if it is permitted to accumulate. If, however, this piece of scrap is picked up and sold to a scrap dealer, it immediately becomes a source of raw material to the steel industry. The scrap dealer will start the ball to rolling by preparing it for mill consumption and he will lose no time loading it in the car and shipping it to some mill. The mill, I assure you, will lose no time melting this piece of scrap and manufacturing it into something very useful to the economy of this country.

Most people are not aware of the fact that of all the new steel produced in this country, approximately 50 per cent is produced from scrap. So you may look at it this way, as an example. The springs in the bed on which you slept last night were produced from 50 per cent scrap. The car you drove to this meeting, this building, contains steel which was produced from 50 per cent scrap. I mention these everyday things to emphasize the fact that all of us feel the influence of steel produced in our everyday existence. Therefore, when there is a shortage of raw material, scrap, all of us are affected.

Whenever there is a crisis, shortage, or something of that nature, it is natural to look around for someone on whom to place the blame. A current popular thing to do is place the blame on the Government. Many people would like to place the blame on the scrap industry, others will point at the steel industry itself and say "there is the cause of our trouble". Actually, the shortages are caused by a combination of circumstances. First, let us look at the tremendous expansion that has taken place in the

steel industry. In 1950, the industry produced 98 million gross tons of ingots. In 1951, it produced over 107 million gross tons. In 1952 it is estimated that it may produce over 115 million gross tons. Naturally, with these tremendous increases in steel industry capacities, we are going to have an increased demand for raw material. Second, when it became apparent that our natural resources of iron ore were dwindling to a dangerous point, the steel industry set to work to improve the melting practices in open hearths so that more and more scrap could be used and less iron ore. This practice, of course, increased the demand for scrap.

The scrap industry, in general, has not been asleep while these changes have been taking place, and you will note that in many of the towns in Texas you will see quite a bit more activity around the scrap yards than in the past. You will note, in particular, tremendous piles of auto bodies being readied for gigantic presses. You will notice increased facilities for loading regular scrap and, also, more shearing capacity for preparing it for the Mills.

There are three general methods in which we accumulate the scrap for the mills. First, at the mill itself, a certain amount of the steel is returned directly to the furnaces from crop ends and shearings. Second, we get a quick return from large manufacturing concerns, such as the auto industry where a large piece of metal is stamped and the residue is promptly returned to the mill, but a far greater and very important source to us is the purchased scrap which comes from all of the places I mentioned earlier, farms, ranches, etc.

Heretofore the normal amount of purchased scrap was more than ample to supply the demands of the mills, but with all of the increased production and changes in the open hearth practices, it is going to take more than just normal operations in the scrap collecting field.

The scrap dealers themselves have done a marvelous job, but to get us through a crisis this winter, many different forms of scrap committees were formed, and I am happy to say that results of their work have been very

gratifying. The NPA set up a special department for assisting in this program. The Chamber of Commerce have cooperated magnificently, and the steel industry set up a program whereby they utilized some 10,000 steel salesmen who called on all types of industry to urge them to junk their obsolete equipment, clean up their yards and basements, and get this scrap moving to the mills.

Now, the Highway Department has offered to help in any way they can and, in my opinion, this presents a fine opportunity for you gentlemen to augment the work of the other committees in moving this scrap to the mills. In doing this type of work, it will not be necessary for you to spend any great amount of time or detract from your regular work, but a word here and there to farmers, ranchers, business men, etc., who may have accumulations of old equipment or machinery, which will probably never be used again, may influence them to sell to one of the local scrap dealers. Scrap prices today are very high and it is a very good time to sell scrap. A word from you that this vitally needed scrap is so important to the War effort and economy of this country, may influence them to pick up this piece of scrap and send it where it will do some good.

We have dwelled on the importance of steel to your everyday life but in addition to that phase consider what it means to the Highway Department. Each quarter your quotas have been cut on steel for new construction. The raw material or scrap shortages have had as much to do with that situation as any other contributing factor. The scrap is available and if you will look in the places mentioned you will see it in your daily trips in your district. We, therefore, urge you to use your own resourcefulness to encourage the people in your district to help us make this (reinforcing bar) out of this (scrap).

● ● ●

3:00 Contractor's Current Problems
Delbert Ward, President
H. B. Zachry Company
San Antonio, Texas



CONTRACTOR'S CURRENT PROBLEMS
By D.R. Ward

Mr. Chairman, Gentlemen - -

It is a pleasure and a privilege to be invited to speak to you this afternoon.

The highway construction business in particular and the construction industry in general have many pressing current problems that must be discussed, and lines of action determined. In considering these matters it is important that all interested parties participate in solving these problems. The annual Highway Short Course affords an excellent opportunity to present many of these problems.

It is usually boring to have to listen to someone recite and discuss their troubles; however, it is oftentimes helpful to be able to discuss one another's ailments if there is a prospect of some cures being found. With that thought in mind I would like to talk to you about some of the current problems of contractors. In most instances I will try and explain what is being done to solve some of these troubles. The great majority of our common problems seem to defy correction or solution. Most of such problems are the outcome of our current national political situation. The corrective action is glaringly obvious.

The greatest, most painful, and most crippling problem that contractors, businessmen, and you and I, as individuals, have today is the enormous burden of Federal taxation - a large portion of which is concealed and usually compounded. The present corporation income tax is a much greater tax than the British ever dared to impose on the American Colonists, their commerce or industry.

Consider for a moment the situation faced by a certain young engineer who inherited a piece of land, which he sold for \$65,000.00. He decided to become a highway contractor. He started in business with this \$65,000.00

just after the end of World War II. At that time he wisely bought all new equipment, which cost \$75,000.00. He borrowed \$37,500.00; \$10,000.00 of borrowed capital was placed in equipment and \$27,500.00 borrowed capital was used to operate on. In the last five years he has made a small amount of money after taxes in highway construction, but he put it back into his equipment. He now has his equipment worn out and after selling all equipment he has current assets amounting to \$75,000.00 which he must now turn around and spend on a new fleet of equipment if he wishes to stay in business. The cost of equipment has advanced 40% in the last five years and his original list of equipment will now cost \$105,000.00. He has been allowed to charge off only one-fourth of the original cost of his equipment annually from his income tax. He has not been allowed to adjust his tax write-off to take account of the rising replacement costs. He also finds now that he must have operating capital equal to approximately 40% of his fixed capital. Instead of being able to get along on \$27,500.00 operating cash, he must now have \$42,000.00; therefore, he must try to borrow \$72,000.00 to go with his \$75,000.00 in order to replace the equipment and have cash to operate the same business that he started five years ago on \$37,500.00 of borrowed money. Faced with having to bid in work on a fast rising labor and material market, at a profit margin of less than 12% on contract costs, and then faced with paying a maximum of 70% of that profit in income taxes, he realizes that he is in a "no betting limit" poker game with only a dollar cash. He finds his banker won't go with him unless he reduces his operations by at least one-third so that borrowed capital does not exceed 50% of the equipment value. If he had held on to his land, which was advancing in value, he would probably have been worth considerably more today. Now, instead of being able to own, say three, tractors he can only buy and operate two, and his potential profits will dwindle proportionately.

Exactly the same thing holds true for the larger contractors. Inflation and extremely high income taxes are shrinking their operating potential and the value of their capital investment. The solution to this problem is of course lower income taxes and a tax formula that would

permit tax write-offs based on the current replacement value of equipment rather than original cost. In these trying times of vicious taxation and unrealistic controls few new companies can get started and weather the storm. Many old companies are failing. The surety companies advise that the number of contracting company failures in 1951 almost equalled the number of failures in the worst year of the thirties.

The year 1951 was a record year in nearly every field of construction. Total construction amounted to 39 billion dollars. This places the industry at the top of all national industries. The value of all agricultural and livestock sales for the year were $37\frac{1}{2}$ billion dollars.

One of highway contractor's current and ever present problems is how to plan for the future. Some of the questions we try to answer are: What will the size of the '53 program be? Is there going to be an excess of competition in any particular phase of highway construction? Will the program be well distributed over the calendar year? Does the Highway Department have sufficient engineers to do their long range planning and get out the plans and specifications that will insure a continuous highway program? (With a shortage of 60,000 engineers this appears doubtful.) Will there be a shortage of materials, controlled or otherwise, that will delay construction and increase operating costs? What will the cost and supply of labor be?

Railroad cars, particularly gondolas and bottom hopper cars, are in very short supply. Some railroads have done fairly well in handling the situation. One major railroad operating in the area north of Dallas and Ft. Worth has not been able to supply more than half of the demand for such cars. As a result some jobs have been seriously delayed because they could not secure ample shipments of paving aggregates. The solution to this situation is the wider use of long haul truck shipments and a greater use of local materials.

We must design and build our highways to accommodate economical truck transportation. Today 75% of all land freight now moves by truck. It now totals 115 billion ton miles annually of freight. The trucking industry pays over \$250,000,000.00 per year in gas tax and as much or more

in license fees.

One of our problems that causes more concern today than formerly is the transportation of highway construction equipment. It is impossible in 99% of the cases I have noted to move equipment from one project to another entirely by rail. Today, as far as I know, there are only approximately 18 trucks and floats in Texas that can comply with the regulation on number of axles, size of tires, length, etc. required to haul a $1\frac{1}{2}$ yard shovel. At this moment I only know of one trailer in the State of Texas that can legally haul a $2\frac{1}{2}$ yard shovel. Many District Offices will not issue permits for Saturday or Sunday or holiday movements. These regulations and practices work a hardship on the contractor, out of proportion I believe, to the amount of damage that might result.

It is of vital concern to all highway contractors that the financing of highway construction be on a well planned, business-like, secure, and non-political basis. The Federal Government annually collects \$2,000,000,000.00 in gasoline tax revenue. The Government generally returns \$500,000,000.00 to \$600,000,000.00 of this to the 48 States. In order to keep up our primary road system this amount should be greatly increased. The contractors recommend that everyone actively support the Texas Good Roads Association. They are the organization that can best sponsor and promote our interests in these matters on a State level.

The Associated General Contractors of America have always promoted and practiced the principles of free and private enterprise and fair competition. The fullest use of the contract system in performing all construction has always been advocated as the surest way of saving money, of obtaining superior construction, of avoiding graft and political interference. Such agencies as the Bureau of Reclamation, who previously have done as much as 50% of their construction with their own forces, are now limited to 12% by the latest act of Congress. The once popular practice of Municipal, County, State and Federal agencies performing construction with their own forces has invariably proven to be wasteful, inefficient, conducive of graft, and uneconomical. The splendid record of honesty

and fair dealings and the fullest use of the contract system by the Texas State Highway Department is something that all Texans are proud of, and is something that is worthy of emulation by our sister States.

The ever present problem of labor supply and wage rate increases is a current problem that will give us all considerable trouble in the months immediately ahead. The Corps of Engineers and the Bureau of Yard and Docks will spend approximately \$150,000,000.00 in Texas in this calendar year. Each project will specify a minimum wage rate that generally is 15% above what highway contractors are now paying. The demand for labor may exceed the supply during the peak of this military construction program, which will occur in July and August. Wage rates will continue to advance, the Wage Stabilization Board notwithstanding. This, coupled with the recent 6% advance of freight rates, together with the impending advance in steel prices, means higher construction costs in 1952. National highway average costs advanced 12.1% in 1951. They will probably equal this advance in 1952. It is a tribute to the Texas Highway Industry that for the last 6 months prices have ranged well below the postwar high reached in 1948.

We are fortunate that our highway construction industry is operated on an open shop basis. We should strive to keep it on that basis. Why? Because this makes it possible to give the people of Texas the most for their highway dollar.

The problem of obtaining new equipment has been practically nonexistent during the past year. There have been a few critical items such as heavy crawler tractors of one certain make, most makes of crawler cranes, and concrete pavers and asphalt lay-down machines. The demand for these items for off shore military construction has caused a temporary shortage. Repair parts have been generally in short supply for practically all heavy equipment for the last 18 months or more. Little relief is in sight during 1952.

Everyone in the construction industry is having trouble in obtaining construction materials. The controlled materials that we highway constructors are mainly concerned with are steel products, and to a lesser degree copper and aluminum items. It appears now that by the fourth quar-

ter of 1952 the supply of reinforcing steel will about equal the demand. Structural steel shapes, especially wide flange beams, will be in short supply through the second quarter of 1953. I got the impression, while at the Associated General Contractor's Convention in Detroit last week, that the allocation of steel to the Bureau of Public Roads would steadily be increased after the second quarter. We should not experience a shortage of cement this year unless in the Dallas-Ft. Worth area during July, August and September. The supply of other materials should be ample to sustain any highway construction program now planned.

All engineers and contractors are constantly searching for ways to increase efficiency and reduce construction costs. In the matter of shipping bulk cement, the mills will not ship except by rail. In this age of motor truck transportation, it is not generally necessary that concrete batching plants be set up on railroad locations. However, if plants are set up away from railroads then, to use bulk cement, it is necessary to ship bulk cement in box cars or bottom hopper cars, then unload to trucks and haul to the batching plant. Truck transportation direct from the mill is generally cheaper than rail haul up to a haul radius of about 150 miles from each mill. Truck delivery of cement in bulk cement trailers is the ideal way to handle this material in most cases. The highway construction industry should take appropriate steps to break up this log jam. My suggestion would be to work through the Portland Cement Association.

In spite of our problems and troubles, cheap prices and hard competition, the contractors' picture for the next two years looks good. With a good outlook for plenty of future work you will find few pessimists amongst the contractors.

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3:20 Floor discussion on the Above Topics

7:00 Annual Banquet
Sbisa Hall

Toastmaster: D. C. Greer,
State Highway Engineer
Texas Highway Department
Austin, Texas



"A Local Yokel Visits Yurop and the Fur
Yeast"

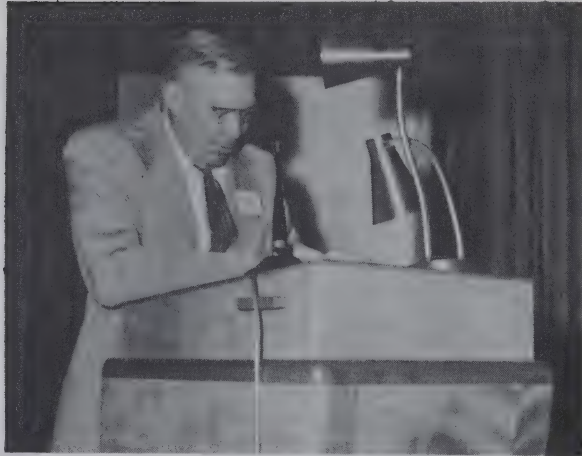
Speaker: W. J. Van London, Engineer-Manager
Houston Urban Project
Texas Highway Department
Houston, Texas



THURSDAY, MARCH 6, 1952

Morning

Chairman: R. E. Killmer, Engineer-Manager
Dallas Urban Project
Texas Highway Department
Dallas, Texas



State Highway Engineer
Texas Highway Department



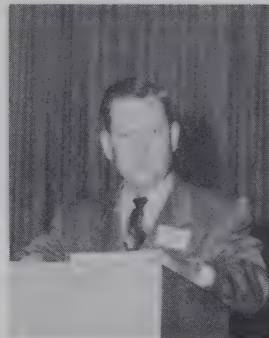
Mr. D. C. Greer

District Engineer
Bureau of Public Roads



Mr. J. M. Page

9:00 Methods Employed in District 7 for Improving Riding Surfaces of Asphalt Roads
R. N. Jennings
District Maintenance Engineer
Texas Highway Department
San Angelo, Texas



METHODS EMPLOYED IN DISTRICT 7
FOR
IMPROVING RIDING SURFACES OF ASPHALT ROADS

BY
R. N. JENNINGS
DISTRICT MAINTENANCE ENGINEER
SAN ANGELO

The methods and operations used in improving riding surfaces in District 7 are no cureall as anyone who has ridden over our pavements can say as we still have rough pavements with us; nor can any improvement be called permanent as highways that rode well after leveling are now rough again due to shifting of base or subgrade. The methods used cannot be claimed as being original with me for most of them were developed by workmen, foremen, inspectors of our district or were lifted from our neighboring districts.

It is desired to break my subject down into four subdivisions -- that is, Why? When? Where? and How?

The "WHY" digresses somewhat from the subject, but the importance of the "WHY" cannot be over-stressed. The easiest attitude for the maintenance department to adopt about a rough section of road is to say, "let it go and when it is real bad we will secure additional funds to rebuild it." This conserves maintenance money and permits maintenance equipment and forces to be placed on other tasks that will make a better showing for the time being. If this attitude is adopted, we are not facing facts, for in District 7, which has comparatively modern highways, we have 150 miles of roads that are over 20 years old and 500 miles that are from 10 to 20 years old. In the 6 years since the end of the war, we have managed to rebuild 54 miles, or an average of 9 miles per year. It is evident that unless the rate of reconstruction is stepped up considerably we are faced with a hopeless task in attempting to correct rough riding roads by rebuilding them.

Fortunately in our district a large mileage of the highways have fair to good alignment, pavement and structure widths, and fairly stable bases, so

that the traveling public will be satisfied for a few more years if we can provide them with a road that does not ride too roughly.

The "WHEN" to improve riding surfaces is easy to answer and that is: ALL THE TIME. However, one is forced to face the fact that other tasks must be performed, that leveling is an expensive operation, and that certain weather conditions are not suited to this type of work. We have adopted the policy of trying to take the backbreakers out of our roads regardless of the cost or how much it inconveniences us in the performance of other tasks. The major portion of the work is done in the spring and summer. If money is available, some missed spots are picked up in the fall. Every effort is made to perform certain types of leveling prior to placing our seal coats so that the appearance of the road will not be marred by the leveling operations.

The "WHERE" to improve riding surfaces is not so easy to answer. One could answer every "WHERE", and I am sure that nearly all roads could be made to ride smoother. However, neither time nor money will permit one to undertake such an all inclusive program.

The backbreaker should be taken out regardless of where it is, as it is not only uncomfortable but hazardous. In selecting the roads to be worked on, consideration should be given to degree of roughness, amount and type of traffic, any programmed reconstruction, and, as has been mentioned, priority should be given to those sections that are to be sealed.

Now after a rather long preamble we come to the subject of this talk. That is - methods of improving riding surfaces.

All of the following methods have been used in our district.

1. Uvalde rock asphalt pavement placed by blading.
2. Blademixes of asphalt and aggregate, which may be either local material or may be crushed Uvalde rock.
3. Spot leveling.

The method selected is dependent on the funds available, on the degree of roughness of pavement, on the time, equipment and forces that may be used for the leveling, and on the amount and type of traffic using the highway to be worked on. No one method may be the answer on a section of road, as a combination of two or more methods may be the better solution.

Uvalde Rock Asphalt Pavement, placed by blading, has been used to level those sections of roads that are near a railhead, thus insuring a fairly economical product, and that ride roughly by reason of a concentration of high and low spot in a limited area. It is felt that a layer of leveling material over the entire surface is the most feasible way to correct this condition.

Type "D" Rock Asphalt Pavement is used for the most part in our leveling. Some Type "C" is used on some sections that demand a thicker layer to accomplish the desired results. The amount used depends, of course, on the roughness of the old surface and on the width of pavement being leveled. It is felt that approximately 4 1/2 cubic yards of Type "D" is about the minimum that can be placed per station satisfactorily on an 18 foot pavement. It is also believed that when more than 7 yards per station are required, it should be placed in two courses to accomplish the best results. If long stretches, with a varying amount of roughness, are being leveled, allowance should, of course, be made in the material placed to care for this variation. It has been found that a smaller amount of material will be required to obtain good results if the larger depressions are taken out by spot leveling prior to performing the blade leveling.

A tack coat is placed prior to spreading the premix on the pavement. Extreme care should be taken to hold the amount of asphalt to a minimum as too heavy a tack will give an excess of asphalt in the leveled section, causing fat spots and in extreme cases shoving will occur. Small quantities of asphalt will give good coverage provided final distribution is obtained by means of the tires on traffic or a pneumatic roller.

The material may be distributed on the pavement in several manners. We find that a hopper box mounted on skids that can be drug by a dump truck and that has an adjustable slot to permit variations in the quantity placed gives satisfactory results. The material is then bladed until its water content has been reduced to a minimum and is in a uniform windrow on edge of pavement. The blade should then start to spreading the material in the thinnest possible layers or "dribbles," each layer being thoroughly compacted with a pneumatic roller and the final layer being flat rolled as well. The results obtained will, of course, be dependent on the care taken in performing the work and particularly

on the skill of the blademan.

Blademixes of asphalt and local aggregates or crushed Uvalde Rock have been used with very satisfactory results. Their use being dependent on a comparison in cost and availability with the Uvalde Rock Asphalt Pavement.

Methods used in placing the blademixes are very similar to those used in placing the Uvalde Premix except that the asphalt must be introduced into the mixture by blading on the road, and the tack coat is placed in half-widths after mixing operations are complete.

A small aggregate is used with 100% passing the 1/2 inch screen, with 40-60 retained on the 1/4 inch, with 75-90% retained on the 10 mesh and 90-100% retained on the 20 mesh sieve. We have used R-C-2 and MC-3 with the local aggregates and RO-3 with the Uvalde Rock. All gave satisfactory results. One needs warm weather to be able to obtain good results in blademixes.

The amount of asphalt used per cubic yard is dependent on the grading and characteristics of the aggregate. We have found that approximately 15 gallons per cubic yard works for our aggregates. However, the asphalt is introduced cautiously in three or four increments, with a thorough mixing by blades after each increment. The inspector determines the final amount added on each mixing land, basing the quantity on the appearance of the mix. Light colored spots in the land have an additional "shot" of asphalt so that the entire land will present a uniform color after mixing. A rotary broom is used in conjunction with blades to keep the fines incorporated in the mix and every effort is made to speed the processing to prevent the asphalt from becoming tacky before it is laid in. It is spread and rolled in much the same manner as the premix material. After spreading and pneumatic rolling operations are complete, a covering of uncoated aggregate is spread over the mixture at the rate of about 1/4 cubic yard per station. This material is then thoroughly rolled and a certain percent of it will go in voids of the mixture and stick, but its main purpose is to keep traffic tires from coming in contact with fresh mixture and flushing any asphalt to the surface before it has a chance to become entirely stable.

The fourth method of leveling employed is called by us "SPOT LEVELING" and gets its name from the fact that the leveling is done in spots rather than over a large area. The spots may be very close together or may be very scattered,

LEVELING WORK IN DISTRICT 7



Depression located and marked by stretching string across it.



Depression outlined. **Note** cracks fill by squeegeeing asphalt into them.



Depression tacked lightly. Emulsion used here.



Spreading and leveling asphaltic concrete over tacked areas.



Spreading roadmix on tacked surface.



Work checked after rolling to insure smooth riding.

depending on the degree of roughness of pavement. We feel that the maximum results in money and effort are attained by this method. Although the high use of labor will run the cost per cubic yard placed up, a crew that used good judgment in selecting the spots to be leveled can obtain excellent results by using very little material as it is all placed on the spot causing rough riding and none is used to cover pavement that is riding satisfactorily.

The general location of spots to be worked on may be selected by riding over the road or by observing the spring action of vehicles using the road. A string is used to localize the low spots by stretching along the road. A string of sufficient length to cover a hundred or more feet should be used as too short a section stringlined may result in some of the cause of roughness to be overlooked. The low spots shown by the string should be outlined by paint or asphalt and no effort should be made to make a regular shaped patch as very few holes come in regular shapes. The odds are that a rectangular or square patch will ride rough.

The spot to be leveled should be tacked lightly and that "lightly" is important since too heavy a tack will come through a patch and seal coats for years. In some cases, if the pavement is rich in asphalt, the best tack coat may be only kerosene. A mixture of kerosene and cutback asphalt makes a good tack if used in limited quantities. We primarily use emulsions for our tacks because it is felt that a mixture of emulsion with a variable quantity of water will give a tack to meet the varied asphalt conditions found in old pavements.

After the tack coat has become tacky, it is covered with the leveling material, which for the most part in our district is Type "D" Uvalde premix. The material is spread out by blading or raking and is checked with a straight-edge to insure that it is not too high and that it featheredges out to the old pavement. The material is then rolled and again checked with a straightedge to see that no low spots remain. If some remain, additional material is added and the process repeated until the desired results are obtained.

Good workmanship is constantly stressed for poor work will not only probably cause the road to ride rougher but will have thrown quite a sum of our scarce dollars down the drain with nothing to show for them.

Certain operations are sometimes required before leveling is started.

Every effort is made to seal cracks before placing the leveling material. We undertake to accomplish this by flushing asphalt into the crack by squeegeeing the asphalt into the crack. It is felt that unless this is done the crack will soon work its way up through the leveling course.

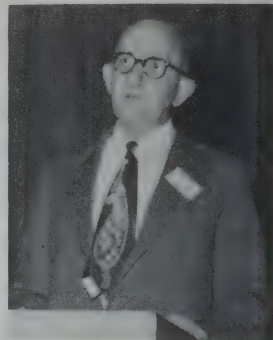
Where volume of traffic requires it, efforts are made to increase pavement widths at the same time that leveling is being done by means of blade operations. This will sometimes mean that additional base material must be placed. Where pavements are widened, a prime coat should be placed on the widened portion with a slight overlap on the existing asphalt to provide as good a bond as possible.

The methods used for improving riding surfaces are a matter of opinion and will and should vary from district to district, but by using good workmanship and by keeping eternally at leveling operations, it is felt that our rough pavements will improve in riding qualities, or at least will not deteriorate to such an extent that reconstruction will be required.

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9:25 Methods of Widening and Resurfacing Old
Concrete Pavements
T. S. Huff, Assistant Engineer Road Design
Texas Highway Department
Austin, Texas



METHODS OF WIDENING AND RESURFACING OLD CONCRETE PAVEMENTS

We entered the War in 1941 with a concrete paved road system of approximately 5000 miles. Of this total, 94% was twenty feet wide or less and the entire system, almost without exception, was six inches thick regardless of whether it had been constructed on good subgrade or on weak plastic soils. Furthermore, about 80% of the concrete roads at that time had been constructed in the decade immediately prior to the War's beginning. These roads were an average of five years old and, judged by the standards of that time, were adequate. They had been designed for pre-war wheel loads and the older pavements had performed successfully under these loads.

When the War began we were called upon to help the railroads handle the heavy transportation connected with supplying the Army and Navy. All load restrictions were removed. The only restrictions left in force were those making it impossible for us to properly maintain the highways. As a result the concrete pavements on the arterial system and those near war activity centers were a shambles at War's end. A system of roads, which under normal pre-war usage would have reached its life expectancy of thirty years, was virtually destroyed before it was ten years old.

Before 1945 our design thinking, insofar as rehabilitating old concrete pavements was concerned, was limited to the addition of a concrete strip on one or both sides of the old slab and subsequently covering both the old and new work with a thin layer of asphaltic concrete. The widening was simply a safety measure and the resurfacing was provided to make a smooth riding surface. After 1945, when it was realized that the necessities of war had caused a serious deterioration of our concrete pavements, our engineers recognized the need for providing stronger pavements as well as wider, smoother pavements. This theory was supported by practices reported from other states where much thicker overlays were being constructed than had been previously constructed in Texas. In this way the stage was set for the use of thick overlays and the use of asphaltic concrete for widening. The first major project of this

type was constructed soon after the War on one of the interstate routes in southeast Texas.

Since these early days after the War, almost every imaginable method of rehabilitating old concrete pavements has been attempted. They have been widened with Portland cement concrete, with flexible base and many miles have been widened with hot mix asphaltic concrete. These three types of widening have been used indiscriminately in the several climatic and soil belts of the State. For surfacing, asphaltic concrete has been used almost to the exclusion of all other types of material. The exception has been the use of flexible base overlays on some four or five projects. The most popular of the asphaltic concrete materials, as you know, has been and still is hot mix. The lack of variety in types of resurfacing is more than compensated for with variety in thickness and method of application. Projects already constructed and plans for projects now being processed specify thicknesses all the way from one inch to five inches; binder course types range from B and C to E. One designer prefers to have all courses the same type while another likes to have them different; still another designer says that if he can't get but one or two inches of thickness he had as soon have nothing, while his neighbor may tell you that one inch will do anything that six inches will do.

We have, until now, explained our confusion away by considering the past few years as having been the evolutionary or experimental phase of this particular subject of road design. Every project we have built can be considered a test project. By the proper evaluation of all conditions and by seriously studying these test projects we should be able to design and construct better and more economical jobs. The old question "What is a good buy?" should come strongly into play. Should the overlay be two, three, four or five inches thick? Remember one inch added thickness costs \$5500 per mile and for a 5000 mile system it amounts to $27\frac{1}{2}$ million dollars.

It is also recommended that in addition to gaining this and other valuable experience the designer must purge himself of the idea that he is a "concrete man" or an "asphalt man" or a flexible base man". If he will let the laws of engineering, economics and above all common sense have free play the material situation will take care of itself.

There appears to be a very small area of complete agreement among engineers regarding this subject. Points on which we agree can be listed as follows: (1) all cavities under the old slab should be filled by undersealing, (2) all loose slabs should be removed, and (3) a very light tack coat should be placed on the old slab.

The points of disagreement are many and often a good case can be made for either side. For example, the proponents of the large aggregate binder course point to the theory that the large aggregate will bridge the cracks and joints and create load transfer properties thus reducing cracking through. The advocates of the sheet asphalt binder course point to the sealing and re-sealing properties of that product as a means of reducing cracking through and sealing the water from the subgrade. Claims and counter claims are made regarding such points of design as types of material for widening; whether to widen on one side or two; whether to underpin and reinforce widening strips; types and thickness of overlay material. Other points of disagreement are too numerous to detail here.

There are two reasons for this diverse opinion, (1) conditions are often examined only superficially and (2) the designing engineers in the field are getting too much conflicting advice from several sources of information. No doubt excellent advice is carried in stock by representatives of these various sources, but often the interests of the several groups run counter to each other and it is only human nature for a person's interest to influence his judgment. If the designer gets too confused, he must turn a deaf ear on this army of advisers and return to his own inner knowledge of highway engineering in general and to several years experience in this particular subject.

I will now discuss the subjects of widening and resurfacing separately and in more detail.

Almost all types of widening have performed satisfactorily except a few cases in which some of the basic principles of highway engineering were either ignored or forgotten. For example, a recent hot mix widened job developed typical flexible pavement edge failures. It had been constructed in a trench on one of the weaker soils in a wet country in violation of a principle we have all respected for many years.

If this fifteen inch hot mix strip had been placed on a well drained subgrade, and had the shoulders been made of granular material, it is almost certain that it would not have failed. There is also reason to believe that it would not have failed if it had been made of concrete and had been underpinned and reinforced. If that be true, then lets cooperate with the laws of nature and use the various materials at our disposal where they ought to be used.

Generally, I think it will be found to be more economical to widen concrete with concrete in the highly plastic soil belts and with hot mix asphaltic concrete in the sandy soil belts. The latter type is particularly recommended where local hot mix materials are abundant.

Whatever type of widening material is used it is recommended that the work be done on one side of the road only. For concrete strips greater bearing area is provided and one longitudinal joint is eliminated. For flexible widening the possibility of edge failure is reduced by half. Furthermore it is a fact that the traffic path will distribute itself over a wider area if one strip is used, thus reducing the number of loads per unit of area. These reasons appear to be so important to the maintenance and operation of the road that widening on two sides should be completely abandoned.

There have been enough cases of upward faulting of concrete widened strips in plastic soils to warrant underpinning and reinforcing. No appreciable trouble of this kind has appeared in sandy soils, but there have been enough reports of pulling away of the strip to warrant underpinning and reinforcing of all concrete strips.

When the plans call for covering the old and new work with as much as three inches of overlay material it is recommended that the strip, if it be made of concrete, be constructed without forms and that all joints be eliminated. In addition to the reduced cost, there will be less interference to traffic.

If flexible base material is used for widening, it is recommended that the same material be used to blanket the entire roadway including the shoulders. A one inch binder course of Type E hot mix placed directly on

the slab is suggested as waterproofing and to prevent the base material from leaking through the joints and cracks.

Regarding the subject of resurfacing there are three possible benefits to be derived, namely:

1. A smooth riding surface. (Impact stresses which are known to accelerate faulting and cracking are reduced)
2. A more waterproof subgrade. (A more constant moisture content reduces the peaks in the wet-dry cycle. Weakening of subgrade through overwetting and cracking through overdrying is reduced.)
3. A pavement of greater structural ability than the one being replaced. (The ideal design will equal that of a new pavement providing for present and anticipated loads.)

These three things are about all that can be done with an old concrete pavement. The ideal job would accomplish all three objectives. In so doing an old broken up structure which has failed under the poundings of fifteen to twenty thousand pound wheel loads would be transformed into a smooth, waterproofed structure capable of handling these same loads.

How near are we coming to accomplishing this ideal in our present resurfacing practice? The requirements for new design on the weaker soils are ten to twelve inches of concrete or twenty to twenty-five inches of flexible pavement. If we are honest with ourselves, we will admit that we have not built one resurfacing job that will equal either of these new design requirements.

Under the present resurfacing procedure we are, then, providing some added strength through reduction of impact stresses and waterproofing of the subgrade. Almost no additional basic structural ability is being provided by the addition of even four or five inches of hot mix and probably very little when as much as eight inches of flexible base material is added. It is doubtful whether we can afford to add flexible pavement depths almost equal to new pavement requirements in order to make the resurfacing job equal to a new one. There would be other serious problems to arise if these depths were added; namely, a too high grade line and the necessity to adjust ditch grades.

It is, therefore, recommended that we recognize our inability to economically provide flexural strength to our worst pavements and concen-

trate on either making them as smooth and as waterproof as possible or taking them out. We have had valuable experience in the former.

It has been found that a one inch thick binder course of Type E hot mix, if placed directly on top of the old pavement with a machine, will provide maximum waterproofing. The level-up course should be placed directly on the binder course and spread with a blade. The one inch thick surface course should be machine placed. By using a maximum of about three inches of hot mix in the binder course, leveling course and surface course, everything is done for an old pavement that can be done unless enormous overlay depths are used. Admittedly it is a "shot in the arm" treatment but it will prolong the life of the old pavement about as much as some of the intermediate depths which also fall short of providing adequate flexural strength.

There are many examples of successful projects being resurfaced with less than three inches. One such job has been giving good service since 1938. Concrete strips were placed on both sides and they were underpinned and reinforced. The overlay depth was about two inches thick. The job is in the wet lands of southeast Texas and during the fourteen years of its life it has had very few failures. It is scheduled to receive another thin overlay this year, the first it will have had since the original resurfacing was done. There are several other jobs five and six years old with two inch thick overlays which are giving good service.

While discussing individual projects I would like to call your attention to a wire mesh reinforced hot mix project constructed in Liberty County on U. S. Highway 90. Its behavior after five years, compared to non-reinforced projects on the same highway in that vicinity, is highly favorable.

Also I would like to comment on the sequence of constructing widening and resurfacing. Insofar as I can determine almost all the successful thin overlay jobs were resurfaced immediately after the widening was done.

There are some thin overlay jobs which are failing. However, it is my opinion that the successful jobs far outnumber the failures. When a job fails, it only proves that the "shot in the arm" treatment is not enough. It is probable that a combination of extremely heavy loads, badly broken pavement and weak subgrade exists. In such cases we should either (1) re-

move the old pavement and replace with new, (2) construct a new road alongside the old and remove half the traffic or (3) put the old pavement on modified service. The latter solution will require a relocation and relegation of the old highway to farm road status.

Before getting away from the subject of design I would like to touch briefly upon the material composition of the various widening and resurfacing methods. The merits of Type E hot mix for the binder course has already been pointed out and cannot be stressed too strongly. The other layers, and particularly the widened strip, when made of hot mix should be made of local materials if possible. The widening when made of concrete should also take into account local material sources. Particular investigation should be made of shell and sand aggregates available along the Coast. Above all I think our best engineering can be accomplished by the proper selection and use of local materials. Sometimes the ideal solution may well be sacrificed if enough money can be saved.

Before closing I would like to stress the continued importance of this subject. Exclusive of the farm roads, 20% of the state highway system is made of concrete pavement. These concrete roads carry 31% of the traffic, which is another way of saying that 31% of the gasoline tax revenue is earned on concrete roads. It is good business to keep this important money making increment of our highway plant in good condition.

I must report to you that we are far behind in this task. In spite of all the work we have done, more than two-thirds of the concrete pavements are twenty feet wide or less. The job we started when the War ended is less than one-third finished. It is my opinion that valuable lessons can be learned from what we have done and if the information is intelligently used in the remaining work a better job and a more economical job can be done.

• • •

9:55 Thoughts and Theories on Penetration Surfaces

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THOUGHTS AND THEORIES ON PENETRATION SURFACES

PRESENTED BY: JEROME P. KEARBY, SR. RES. ENGINEER

TEXAS HIGHWAY DEPARTMENT, DISTRICT 2, FORT WORTH, TEXAS

TWENTY-SIXTH ANNUAL HIGHWAY SHORT COURSE

TEXAS A & M COLLEGE, MARCH 6, 1952

* * * * *

This subject of Penetration Surfaces has developed many controversial ideas, and it is not our desire to add to these; nor do we desire in any way to belittle or cast any reflection upon any ideas, as we realize that they all have their merits, and have been more or less satisfactorily used.

We are indebted to Messrs. G. M. Garrett, W. W. Finley, R. J. Hank, J. C. Roberts, Marshall Brown, H. C. Carter, A. J. Van Dyke, and others too numerous to mention, for their outstanding work in experiments, tests, theories, and papers on this subject, which have developed a background for our theories.

The writer is also indebted to Messrs. J. R. Mercer and H. A. Auvenshine for their able assistance in preparing this paper.

We feel that we are in need of design procedures for Asphalt Surface Treatments, which have the advantages of simplicity and uniformity by which the required amounts of asphalt and aggregate may be determined, by test methods, rather than left entirely to visual approximation and judgment.

In recent years, and at the present time, we are hearing much about the hazards and damage caused the traveling public due to flying stones, dust, and slick asphalt surfaces.

We have endeavored to reduce these conditions in every way possible, and have observed that in using the coarser grades of aggregate of nearly uniform size, with a spread ratio just sufficient to slightly more than cover the surface one stone in depth, and by limiting the asphalt to an amount sufficient to embed only a portion of the thickness of the loose mat of aggregate, we have most nearly approached the desired results.

We have observed that the specification requirements for gradations of aggregates are very broad, permitting coarse graded aggregates, fine graded aggregates, and aggregates graded fine to coarse, to be furnished within the limits of each grade. These aggregates

when carefully analyzed, vary considerably in particle sizes and shapes, as well as percent of voids.

It is apparent that the particle sizes and shape of aggregates control the amount of a particular aggregate needed as sufficient cover material, and that the amount of asphalt required is controlled by the sizes and shape of particles and percent of voids in the aggregate, as well as the condition of the base or surface which is to receive the application.

Design procedures are desired by which the required amount of asphalt and aggregate can be predetermined that satisfactory surfaces may result.

There are a number of basic types of asphalt surfaces and pavements, and numerous variations in materials and construction methods have been more or less successfully used in the construction of these different types. The selection of type, as well as the materials and construction methods, may therefore, be governed by a number of local conditions including: volume and weight of traffic, characteristics of roadbed, characteristics of available mineral aggregates and asphaltic materials, available construction equipment and last, but not least, cost.

The term "surface treatment" is ordinarily used to describe the application of an asphaltic binder with a cover coat of mineral aggregate to produce an asphaltic mat of appreciable thickness on a prepared base. The thickness of a one course surface mat is ordinarily controlled by the average size of aggregate particles in the cover coat. Sometimes, multiple surface treatments consisting of two or more applications are constructed to increase the density of the surface.

A surface treatment called the seal coat is frequently applied to an existing surface to make the surface waterproof or non-skid. Seal coats are most frequently used to replace or rejuvenate old, dried and worn out bituminous surfaces and/or to provide non-skid surface on old, slick asphalt surfaces.

This paper is devoted primarily to a recommended practice in determining the amounts of asphalt and aggregate required in the construction of one course asphalt surface treatments, and seal coats.

The succeeding suggestions for asphalt surface treatments and seal coats are recommended for use under what may be termed general and average conditions. It is realized, however, that no single standard procedure will cover satisfactorily all variations in local conditions, which may prevail for individual jobs.

The quantity of aggregate and asphalt required for a given area varies according to the character of the aggregate, being influenced by the unit weight, percent of voids, shape and size of the particles. Thus, as a rule, somewhat less weights of slag and certain gravels are required for a given amount of asphaltic material, as compared with most grades of broken stone. The engineer should modify the amount according to local conditions. For surface treatments it is always better to err on the side of a slight deficiency of asphalt so as to avoid a fat, slick surface. Our first and last thought should be to avoid the hazard to traffic created by flying rock particles, and the danger to traffic created by fat, slick surfaces.

It is our opinion that the aggregate in an asphaltic surface treatment should be sufficiently embedded in the asphalt to resist displacement, but the asphalt should not be so close to the surface after compaction as to permit tires to reach and spread it.

During the past several years, much time and thought has been devoted to the construction of satisfactory asphalt treated surfaces. Much valuable information has been derived from these experiments and studies, but heretofore we have known of no standard and reliable set of procedures by which the quantities of asphalt and aggregate can be predetermined. With this in mind, we have endeavored to devise a set of procedures which will reasonably assure economical asphaltic treated surfaces with the following characteristics: a smooth-riding, non-skid waterproof surface of uniform texture and color, which during construction will minimize dust and flying rock particles.

In devising such a set of procedures, it is the writer's opinion that the quantity of aggregate should be that amount of aggregate that is required to form a blanket of aggregate one stone in depth. That is to say that each stone covers some portion of the area; therefore, the quantity of aggregate is definitely fixed and determined by its unit weight and the shape and size of aggregate particles. Also, the quantity of asphalt is dependent upon the percentage of voids in the aggregate; the desired depth of aggregate to be embedded in the asphalt, and to some extent, the type of surface or base on which the treatment is to be placed, as well as the hardness of the cover stone, and the type, kind and amount of traffic.

The following are the main factors involved in predetermining the amount of aggregate and asphalt required for satisfactory asphalt surface treatments:

First: The amount of a given aggregate required to cover one square yard in area, which we have termed the "Effective

Mat Thickness," or average thickness of the aggregate when placed so as to form a blanket of aggregate one stone in depth. (See Figure I - Aggregate Size).

Second: The amount of asphalt per square yard of surface that will be required to embed a desired portion of the "Effective Mat Thickness" of the aggregate. (See Figure II-Req. Asphalt).

The following procedures were used in determining the aggregate factors: The unit weight, specific gravity, voidage and screen analysis of the aggregate were determined.

The effective mat thickness, being dependent upon the screen analysis of the material being used, we set up the following sample procedures to compute each of the above factors for a sample of Grade 1 aggregate, Item 351 of the Texas Highway Department Specifications.

From an individual screen analysis of the aggregate, a middle specification gradation was obtained, assuming the rock size to be the average between the passing and retaining screen sizes. (See Figure I - Aggregate Size).

Using the middle gradation, the effective mat thickness of the rock blanket was computed as indicated on Figure I.

Figure I is a breakdown of Grade 1 aggregate, Specification Item 351, using a sample that fits the middle of the gradation requirements. You will note that the specification gradation has been reproportioned to obtain an accumulative average gradation, showing the percent retained on each screen size within the specification limits, and we have indicated the average individual screen analysis, the sizes of each being that which is half way between the passing and retaining screen sizes. The average particle size or effective mat thickness is determined by multiplying each individual screen size by its percentage, and then obtaining the sum of the products. The theoretical spread ratio equals one yard, or 36", divided by the effective mat thickness; the quotient being the number of square yards spread for each cubic yard of material. This method of determining average aggregate particle sizes, theoretical mat thickness and spread ratio for a cover of one stone average thickness, can be satisfactorily used if flat and elongated aggregate particles are not present.

When flat or elongated aggregate particles are present, the screen analysis method of determining average particle size, effective mat thickness, and spread ratio cannot be

Asphalt Surface Treatments

Method Used In Computing Average Agg. Sizes Using Grade I Average Aggregate

Sizes	Percent Retained On									Percent Passing	
	1"	7/8"	3/4"	5/8"	1/2"	3/8"	1/4"	#10	#20	#10	#20
Specification accumulative gradation											
Percent			15/45		85/100		95/100	97/100		0/3	0/3
Average accumulative gradation											
Percent	0	15	30	61	92.5	95	97.5	98.5	99	1.5	1

Sizes	Average Individual Screen Analysis									
Avg. Sizes	7/8"-1"	3/4"-7/8"	5/8"-3/4"	1/2"-5/8"	3/8"-1/2"	1/4"-3/8"	#10-1/4"	#10		
Percent	=15/16	=13/16	=11/16	=9/16	=7/16	=5/16	=3/16	=1/16		
	15	15	31	31.5	2.5	2.5	1.5	1.0		

Calculations For Average Particle Sizes Of Aggregate And Effective Mat Thickness

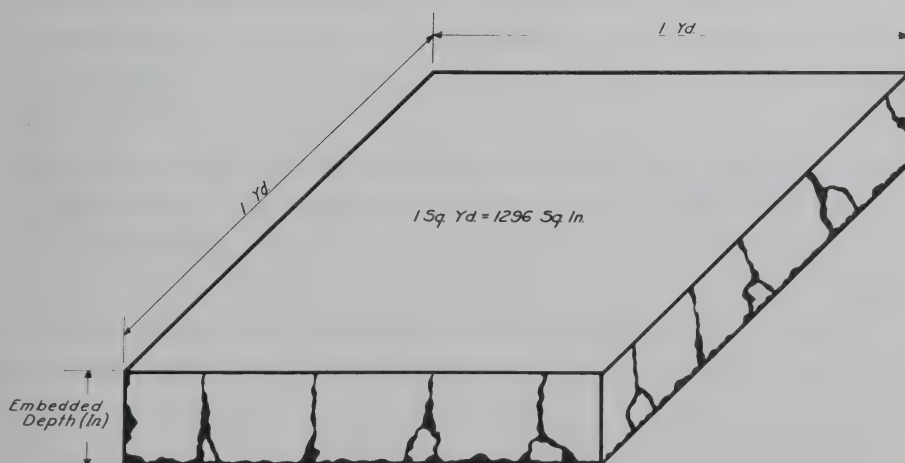
Percent Times Average Size
 $15\% \times 15/16 = 2.250/16"$
 $15\% \times 13/16 = 1.950/16"$
 $31\% \times 11/16 = 3.410/16"$
 $31.5\% \times 9/16 = 2.835/16"$
 $2.5\% \times 7/16 = 0.175/16"$
 $2.5\% \times 5/16 = 0.125/16"$
 $1.5\% \times 3/16 = 0.045/16"$
 $1.0\% \times 1/16 = 0.010/16"$

 $100\% \text{ Total } 10.800/16" = 43.2/64"$

The average size of Aggregate particles being $43.2/64"$
The Theoretical Mat thickness for one stone cover = $43.2/64"$
The Theoretical Spread Ratio = 36 divided by $43.2/64" = 1:53$
or 1 cu. yd. of Aggregate for each 53 Sq. Yds. surface.

Fig I

Asphalt Surface Treatments



Method Of Determining Amount Of Asphalt Per Sq. Yd.
Having determined % Voids in aggregate and
average Mat Thickness and selected the desired %
Embedded.
 $\% \text{ Embedded} \times \text{average Mat Thickness} = \text{Embedded Depth.}$
 $\text{Asphalt Volume in gals per sq yd} =$
 $1296 \times \text{Embedded Depth} \times \% \text{ Voids} =$
 $\text{Cu. In. of Asphalt} = \text{Gallons Required}$

Fig II

depended upon, as the flat and elongated particles will tend to lie on a flat side and thereby reduce the effective mat thickness, cover a greater area, displace other particles, and consequently increase the spread ratio.

For most aggregates, and especially for aggregates containing flat and elongated particles, the square yard test board should be used to determine the average particle depth, effective mat thickness and spread ratio, the procedure being as follows:

Obtain a one square yard test board. Place a sufficient quantity of the aggregate on the board to obtain a full coverage one stone in depth. Weigh the aggregate required for the one stone depth coverage. Then, the pounds per cubic yard divided by the pounds per square yard equals the spread ratio in square yards per cubic yard. The average particle depth and effective mat thickness is obtained by dividing 36 inches by the number of square yards covered per cubic yard.

Figure II is a graph showing the displacement of asphalt due to the embedding of aggregate. You will note that the method used to determine the amount of asphalt required is dependent upon the percent of the average rock mat thickness to be embedded in asphalt, and the percent of voids in the aggregate.

The desired percentage embedded times the average rock mat thickness equals the embedded depth.

One square yard, or 1296 square inches times the desired embedded depth, times the percent of voids in aggregate equals volume of asphalt required per square yard, based on the hard grades of oil asphalts containing approximately 99% bitumen.

This calculation is not precise, in that the percent of voids in the embedded portion of the aggregate mat is not necessarily the same as in the aggregate sample. The percent of voids in the embedded portion of the aggregate mat can be determined by laboratory tests; however, we have observed that in using the percent of voids in the aggregate sample as a factor in the design procedure, satisfactory results have been consistently obtained. If liquid asphalts containing volatiles or emulsion asphalts containing water are used, then the quantity should be increased proportionately to their bitumen content.

Figure III is a table showing the variations in gradation as permitted under the specification, divided into three categories: the coarsest grade, middle grade and finest

FIGURE III
TEXAS HIGHWAY DEPARTMENT

SPECIFICATION ITEM 351
AGGREGATE FOR SURFACE TREATMENTS

TABLE SHOWING VARIATION IN GRADATION AS PERMITTED UNDER SPECIFICATIONS DIVIDED INTO THREE CATEGORIES: COURSE, MIDDLE & FINE GRADE

Percent Retained on Screen Sizes										Percent Passing		Approximate #	Average Voids 35 Percent	
Aggregate Grade	1"	7/8"	3/4"	5/8"	1/2"	3/8"	1/4"	#10	#20	#10	#20	Avg. Spread Size Ratio Inches CY:SY	Recommended Percent Embedded	Required Asphalt-Cem. per Sq. Yd.
No. 1 - Specification	0	-	15 45	-	85 100	-	95 100	97 100	97 100	0 3	0 3	-	-	-
No. 1 - Course Grade	0	45	45	100	100	100	100	100	100	0	0	51/64 1:45	4	0.59
No. 1 - Middle Grade	0	15	30	61	92.5	95	97.5	98.5	99	1.5	1	43/64 1:53	10	0.53
No. 1 - Fine Grade	0	0	15	15	85	85	95	97	97	3	3	35/64 1:65	26	0.39
No. 2 - Specification	0	0	0	-	30 70	-	80 100	97 100	97 100	0 3	0 3	-	-	-
No. 2 - Course Grade	0	0	0	70	70	100	100	100	100	0	0	39/64 1:59	36	0.45
No. 2 - Middle Grade	0	0	0	25	50	70	90	98.5	99	1.5	1	31/64 1:75	34	0.51
No. 2 - Fine Grade	0	0	0	0	30	30	60	97	97	3	3	23/64 1:100	31	0.22
No. 3 - Specification	0	0	0	-	25 50	-	-	97 100	97 100	0 3	0 3	-	-	-
No. 3 - Course Grade	0	0	0	50	50	100	100	100	100	0	0	9/16 1:64	26	0.40
No. 3 - Middle Grade	0	0	0	15	37.5	58	78	98.5	99	1.5	1	27/64 1:84	32	0.27
No. 3 - Fine Grade	0	0	0	0	25	25	25	97	97	3	3	9/32 1:100	27	0.15

Percent Retained on Screen Sizes										Percent Passing	Approximate	Average Voids 35 Percent	
Aggregate Grade	1"	7/8"	3/4"	5/8"	1/2"	3/8"	1/4"	#10	#20	#10 #20	Avg. Spread Size Ratio Inches CY:SY	Recommended Required As- phalt-Cem. per Sq. Yd.	Per- cent Em- bedded
No. 4 - Specification	0	0	0	-	$\frac{5}{20}$	-	-	$\frac{97}{100}$	$\frac{97}{100}$	$\frac{0}{3}$ $\frac{0}{3}$	-	-	-
No. 4 - Course Grade	0	0	0	20	20	100	100	100	100	0 0	31/64 1:74	34	0.31
No. 4 - Middle Grade	0	0	0	6	12.5	41.5	69.5	98.5	99	1.5 1	11/32 1:104	30	0.20
No. 4 - Fine Grade	0	0	0	0	5	5	5	97	97	3 3	13/64 1:178	24	0.09
No. 5 - Specification	0	0	0	0	$\frac{0}{10}$	-	$\frac{60}{100}$	$\frac{97}{100}$	$\frac{97}{100}$	$\frac{0}{3}$ $\frac{0}{3}$	-	-	-
No. 5 - Course Grade	0	0	0	0	10	100	100	100	100	0 0	29/64 1:80	33	0.30
No. 5 - Middle Grade	0	0	0	0	5	42.5	80	98.5	99	1.5 1	11/32 1:104	30	0.20
No. 5 - Fine Grade	0	0	0	0	0	0	60	97	97	3 3	17/64 1:139	26	0.13
No. 6 - Specification	0	0	0	0	$\frac{0}{10}$	-	$\frac{35}{60}$	$\frac{97}{100}$	$\frac{97}{100}$	$\frac{0}{3}$ $\frac{0}{3}$	-	-	-
No. 6 - Course Grade	0	0	0	0	10	60	60	100	100	0 0	11/32 1:103	30	0.20
No. 6 - Middle Grade	0	0	0	0	5	26	47.5	98.5	99	1.5 1	9/32 1:127	27	0.16
No. 6 - Fine Grade	0	0	0	0	0	0	35	97	97	3 3	15/64 1:138	26	0.11
No. 7 - Specification	0	0	0	0	0	$\frac{5}{20}$	-	$\frac{97}{100}$	$\frac{97}{100}$	$\frac{0}{3}$ $\frac{0}{3}$	-	-	-
No. 7 - Course Grade	0	0	0	0	0	20	100	100	100	0 0	11/32 1:107	29	0.19
No. 7 - Middle Grade	0	0	0	0	0	12.5	55.5	98.5	99	1.5 1	17/64 1:133	27	0.14
No. 7 - Fine Grade	0	0	0	0	0	5	5	97	97	3 3	13/64 1:183	24	0.09

Percent Retained on Screen Sizes										Percent Passing		Approximate	Average Voids 35 Percent	
Aggregate Grade	1"	7/8"	3/4"	5/8"	1/2"	3/8"	1/4"	#10	#20	#10	#20	Avg. Spread Size Ratio Inches CY:SY	Recommended Percent Embedded	Required Asphalt-Cem. per Sq. Yd.
No. 8 - Specification	0	0	0	0	0	0 10	-	97 100	97 100	0 3	0 3	-	-	-
No. 8 - Course Grade	0	0	0	0	0	10	100	100	100	0	0	21/64 1:108	29	0.19
No. 8 - Middle Grade	0	0	0	0	0	5	52	98.5	99	1.5	1	1/4 1:140	26	0.13
No. 8 - Fine Grade	0	0	0	0	0	0	0	97	97	3	3	3/16 1:196	24	0.06
No. 9 - Specification	0	0	0	0	0	-	5 35	70 100	98 100	0 30	0 2	-	-	-
No. 9 - Course Grade	0	0	0	0	0	35	35	100	100	0	0	9/32 1:131	27	0.15
No. 9 - Middle Grade	0	0	0	0	0	10	20	85	99	15	1	13/64 1:175	24	0.09
No. 9 - Fine Grade	0	0	0	0	0	0	5	70	98	30	2	5/32 1:230	23	0.07
No. 10 - Specification	0	0	0	0	0	0	2 20	70 100	98 100	0 30	0 2	-	-	-
No. 10 - Course Grade	0	0	0	0	0	0	20	100	100	0	0	7/32 1:169	25	0.10
No. 10 - Middle Grade	0	0	0	0	0	0	11	65	99	15	1	3/16 1:197	24	0.06
No. 10 - Fine Grade	0	0	0	0	0	0	2	70	98	30	2	5/32 1:236	23	0.07

Jerome F. Kearby, Sr. Res. Engr.
James H. Mercer, Jr., Engr. in Charge
Harold A. Owenshine, Draftsman

FIGURE III

grade possible under the specification. For each grade we have calculated and indicated the average particle size and spread ratio, based on screen analysis.

We have also indicated the amount of bitumen required, based on average voids of 35%, and our recommended percent of aggregate to be embedded. Your attention is invited to the possible and probable variation in average size, spread ratio, and required asphalt for each grade within the specification limits, which is still greater when flat and elongated particles are present. Your particular attention is invited to Grade 3 and Grade 4 aggregates which require certain percentages retained on 1/2" screen and #10 mesh sieve, with no requirements specified on the 3/8" or 1/4" screens, thereby allowing aggregates with gap gradations to be furnished.

The quantities indicated in Figure III may be used for estimating purposes in determining the estimated cubic yards of aggregate and gallons of asphalt required for desired grades of aggregate. However, the actual application of asphalt and aggregate will vary with the gradation, unit weight, and percent of voids in the aggregate delivered, and the desired embedded depth of aggregate.

Many Engineers will disagree, at first reading, with the quantities shown in Figure III saying, "In our use of aggregates, Grades 1 through 10, we have used more or less asphalt, and more or less aggregate than shown in the table, and have obtained satisfactory results." This statement may be very true, and it serves to illustrate our point that the rates of application of asphalt and aggregate are dependent upon the average size and shape of aggregate particles, percent of voids in aggregate and characteristics of the existing base or surface. Some of these Engineers may have used a coarse graded material which had a greater effective mat thickness, and greater percent of voids, and thereby allow a greater quantity of both asphalt and aggregate to be used. Likewise, a fine graded material will have a lesser effective mat thickness, and may have a smaller percent of voids, thereby allowing a lesser quantity of both asphalt and aggregate to be used.

Some Engineers may also say, "We have used more aggregate and less asphalt than is shown in Figure III, which resulted in a bleeding or slickened surface." It has been our experience that a considerable excess of cover material is often more detrimental than a slight shortage of cover material, in that with an excess of cover material the amount of fines applied is also increased. The excess of fines tends to go to the bottom and become embedded in the asphalt or blot the surface of the asphalt, thereby preventing the embedding of the coarse material and allowing a large percent of the coarser particles

Asphalt Surface Treatments
 Asphalt Application Rate Chart
 Based on: Aggregate Size, % Voids, and % Embedded

Example:
 Average Aggregate Mat Thickness 1"
 Percent Voids in Aggregate 40%
 Percent Embedded Depth of Aggregate 30%
 Place Straight Edge through Point X and
 40% Void point on 30% Embedded Curve, and
 find Asphalt Application rate to be 0.335 Gal
 per Sq. Yd on 1" Aggregate Curve.

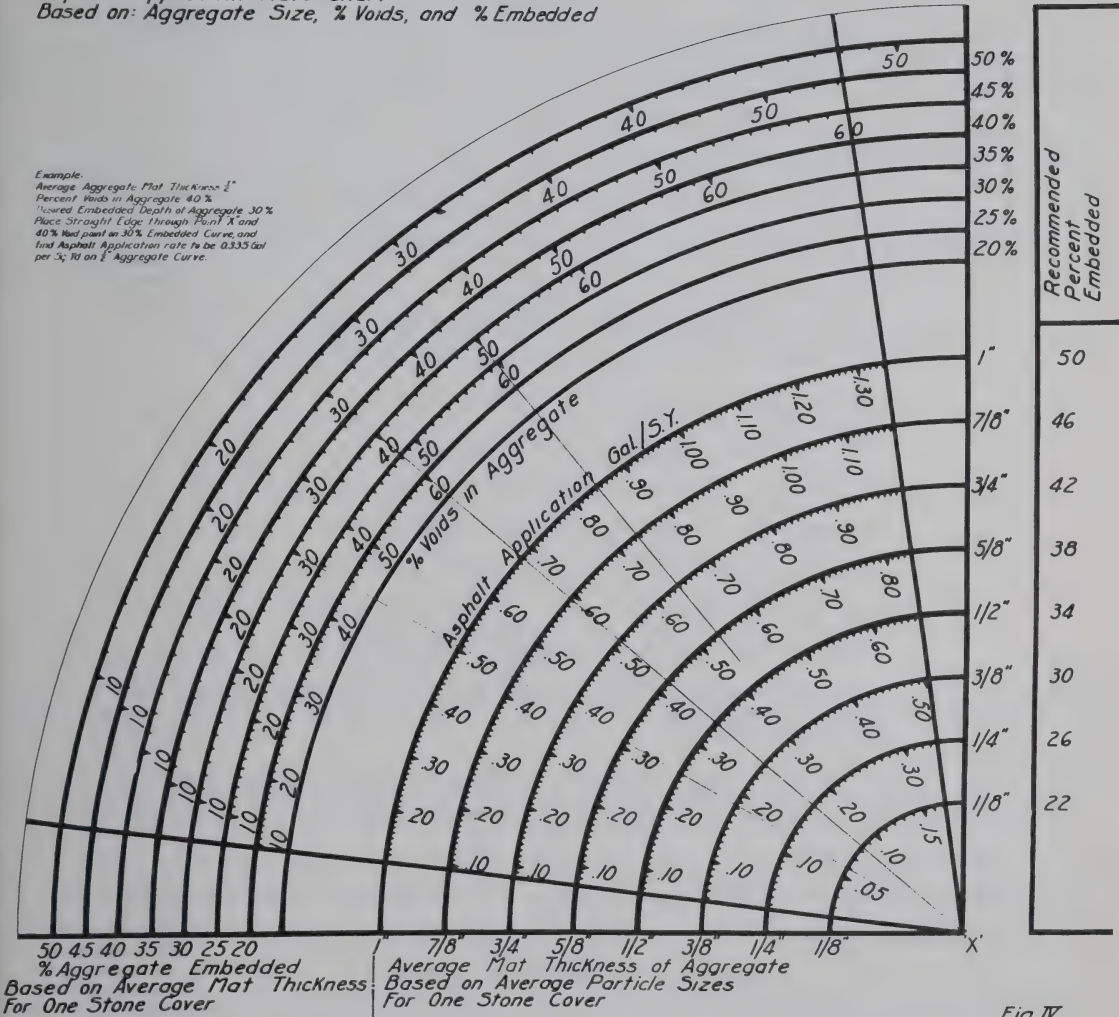


Fig. IV

to be subject to whip-off, leaving a fat or flooded condition.

It is our opinion that the material passing the 10 mesh sieve acts as a filler in much the same manner as sand in a cement mortar, thereby raising the level of the asphalt, and cannot be counted on as cover material to furnish any riding surface.

Figure IV is an Asphalt Application Rate Chart, constructed from mechanical computations as here before explained, from which the gallons of bitumen per square yard can be obtained, after having determined the average aggregate mat thickness, and the percent of voids in the aggregate, and having selected the desired embedded depth. Along the vertical and horizontal axes we have indicated the average aggregate mat thickness for each 1/16 inch thickness up to one inch; also, the percent of aggregate mat embedded for each 5% from 20% to 50%.

On the inner circular lines connecting corresponding average aggregate mat thicknesses, we have indicated the computed gallons of asphalt per square yard for each .01 gallon. On the outer circular lines connecting corresponding percent of aggregate mat embedded, we have indicated the various percent of voids from ten to sixty percent.

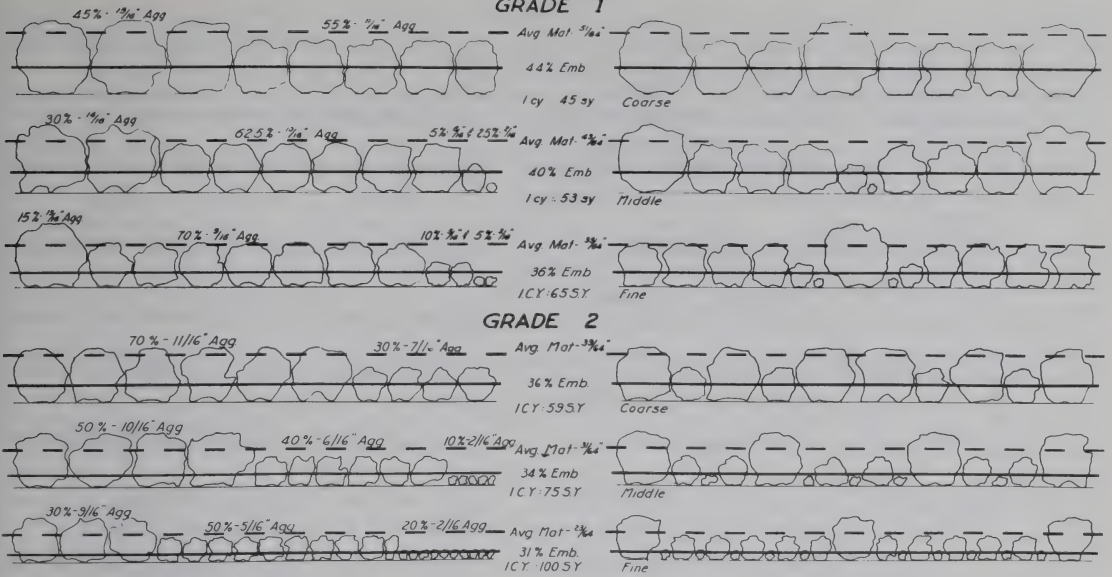
To determine gallons of bitumen required per square yard, follow the desired percent embedded curve to point indicating the percent of voids in the aggregate; connect this point with apex point "X", using straight edge or string; follow along the line so established to curve representing the average mat thickness, and read directly gallons of bitumen required per square yard. Note: If liquid asphalts or emulsions are to be used, the rate of application should be increased proportionately to bitumen content. Your attention is invited to the broad variation in bitumen required, dependent upon the percent of voids which may be as low as 20% or as high as 50%.

It must be remembered that when applying asphalt surface treatments on existing fat asphalt surfaces and on loosely bonded or soft base courses, a portion of the aggregate during rolling will penetrate below the film of freshly applied asphalt, thereby reducing the effective mat thickness of the cover material. Also, that when the softer grades of rock are used, some of the larger sizes will be crushed by rolling, which will reduce the effective mat thickness as well as reduce the voids.

Under conditions as mentioned above, it is recommended that embedding of aggregate be reduced proportionately to the resulting aggregate mat thickness dependent upon the

Asphalt Surface Treatments

GRADE 1

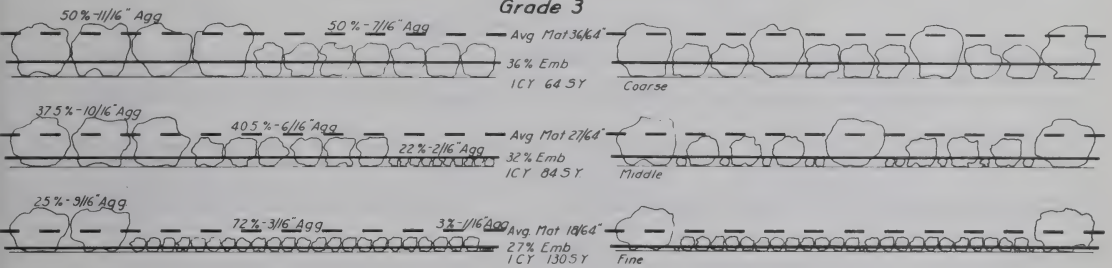


Aggregate Specification Gradation Chart
Showing Average Size & Recommended % Embedded

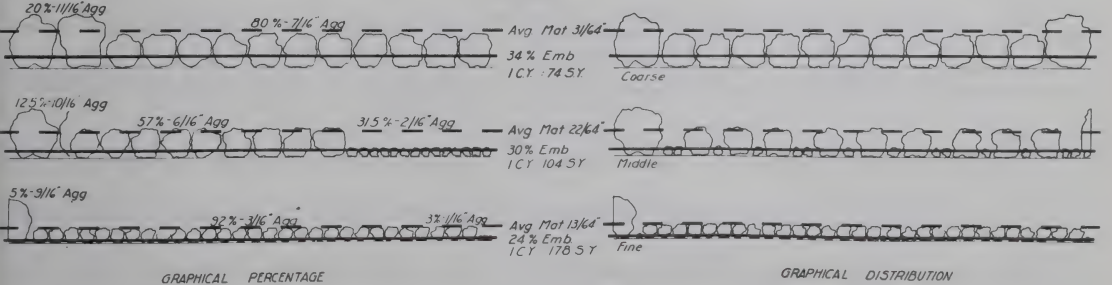
Fig. 1A

Asphalt Surface Treatments

Grade 3



Grade 4

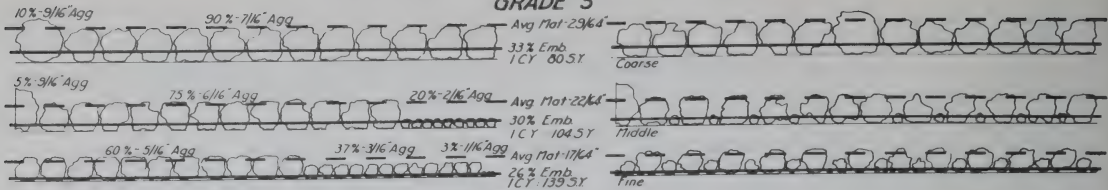


Aggregate Specification Gradation Chart
Showing Average Size & Recommended % Embedded

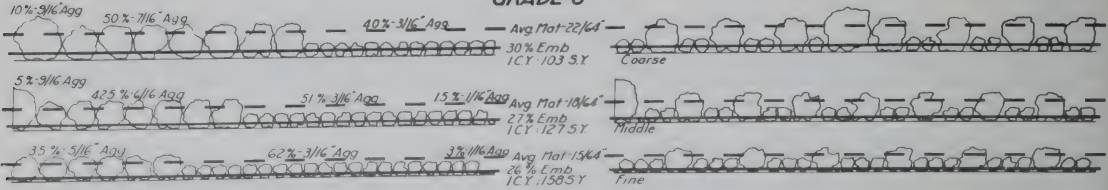
Fig. 1B

Asphalt Surface Treatments

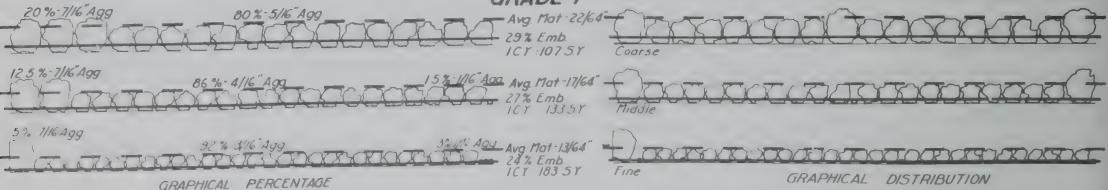
GRADE 5



GRADE 6



GRADE 7

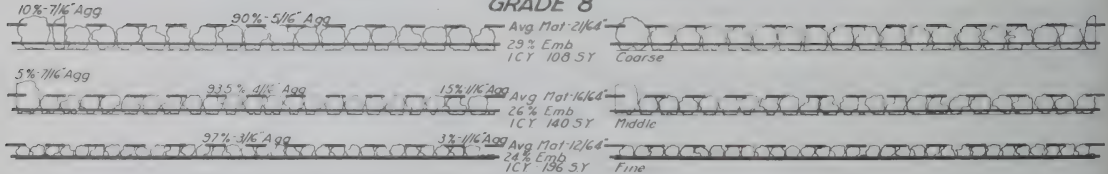


Aggregate Specification Gradation Chart
Showing Average Size & Recommended % Embedded

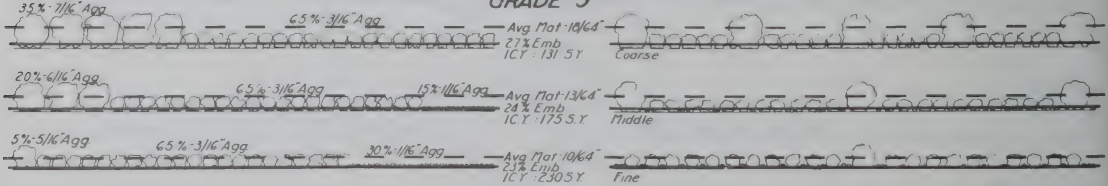
Fig IX C

Asphalt Surface Treatments

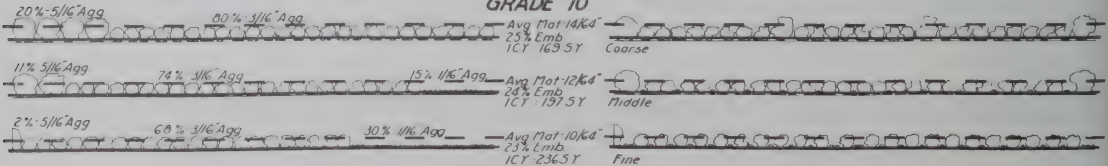
GRADE 8



GRADE 9



GRADE 10



GRAPHICAL PERCENTAGE

GRAPHICAL DISTRIBUTION

Aggregate Gradation Specification Chart
Showing Average Size & Recommended % Embedded

Fig IX D

existing conditions. We know of no way to actually determine to what extent aggregate may penetrate through the freshly applied asphalt and into a loosely bonded soft base, or into existing fat, soft asphalt surfaces, or to what extent the softer grades of rock will be crushed during rolling; however, these factors can be estimated to some valuable degree by placing one layer of the selected cover aggregate on the base or old asphalt surface which is to receive treatment, and observing its condition after thorough rolling with the type of rollers to be used on the job.

When applying asphalt surface treatments to existing hard paved surfaces or tightly bonded hard base courses, it is recommended that the percentage embedded be increased if hard aggregates are used, and reduced if aggregates are soft and break up under rollers.

Some allowances should be made, depending on the amount and type of traffic. For Highways carrying high traffic counts and heavy vehicles, the percentage embedded should be reduced and larger aggregates used. For Highways carrying light traffic counts and light vehicles, the percentage of embedding should be increased and medium size aggregates may be used. .

Figure V shows graphically the gradation and distribution of Specification Aggregates, Grades 1 to 10 - Fine, Middle and Coarse, indicating the percent of various sizes, average aggregate mat thickness, recommended percent of aggregate mat embedded, and spread ratios.

The heavy dash line indicates the top of the average aggregate mat. The heavy solid line indicates the portion of the loose aggregate mat embedded in asphalt.

The percent of aggregate mat embedded in asphalt as recommended herein may appear somewhat light; however, this procedure is based on loose aggregate mats which, under traffic, will change considerably in that aggregate particles will be rearranged and further consolidation will take place, thereby reducing the aggregate mat thickness and percent of voids, and increasing the percent of aggregate embedded. Your attention is again invited to the possible and probable variation in average size and spread ratio within the specification limits, which is still greater when flat and elongated particles are present.

Your attention is also invited to the probable gap grading, especially in the fine gradation of each grade, with a very small percent of the top aggregate sizes, and a large percent of the bottom aggregate sizes. These gap graded aggregates frequently result in unsatisfactory asphalt surface treatments, because of the following:

If the quantity of asphalt used is sufficient to adequately embed and retain the

FIGURE VI

ITEM 351 AND SPECIAL RECOMMENDED GRADING OF AGGREGATES FOR ASPHALT SURFACE TREATMENTS

BY: JEROME P. KEARBY, SR. RES. ENGINEER
TEXAS HIGHWAY DEPARTMENT

* * * *

Grade I	Retained on 1 1/8"	Screen	0%
	Retained on 1"	Screen	40 - 60%
	Retained on 7/8"	Screen	95 -100%
Grade II	Retained on 1"	Screen	0%
	Retained on 7/8"	Screen	40 - 60%
	Retained on 3/4"	Screen	95 -100%
Grade III	Retained on 7/8"	Screen	0%
	Retained on 3/4"	Screen	40 - 60%
	Retained on 5/8"	Screen	95 -100%
Grade IV	Retained on 3/4"	Screen	0%
	Retained on 5/8"	Screen	40 - 60%
	Retained on 1/2"	Screen	95 -100%
Grade V	Retained on 5/8"	Screen	0%
	Retained on 1/2"	Screen	40 - 60%
	Retained on 3/8"	Screen	95 -100%
Grade VI	Retained on 1/2"	Screen	0%
	Retained on 3/8"	Screen	40 - 60%
	Retained on 1/4"	Screen	95 -100%
Grade VII	Retained on 3/8"	Screen	0%
	Retained on 1/4"	Screen	40 - 60%
	Retained on #10	Mesh	95 -100%
Grade VIII	Retained on 1/4"	Screen	0%
	Retained on #10	Mesh	40 - 60%
	Retained on #20	Mesh	95 -100%

AVERAGE SPREAD RATIO FOR SINGLE APPLICATIONS

No. 1	No. 2	No. 3	No. 4	No. 5	No. 6	No. 7	No. 8
1:36	1:41	1:48	1:58	1:72	1:96	1:144	1:288

average particle size, it is not sufficient to adequately embed and retain the top aggregate size. If the quantity of asphalt used is sufficient to adequately embed and retain the top aggregate size, the average particle size will be flooded with asphalt, resulting in an undesirable fat, slick surface.

Figure VI is a recommended grading of aggregates for asphalt surface treatments.

The oversize particles of aggregate permitted by the present specifications are usually the flying stones that we hear so much about as being hazardous and damaging to traffic. The undersize particles of aggregates permitted by the present specifications are often times so fine as to blot the asphalt film and prevent the larger aggregates from becoming embedded in the asphalt.

The present specifications, in many cases allow a gap graded aggregate which is undesirable, and also allow an aggregate graded uniformly from fine to coarse, with a maximum density and minimum voids, which is highly desirable for asphaltic concrete, but not desirable for penetration asphalt surface treatments.

The gradation of aggregates recommended herein will provide aggregates of nearly uniform size, which can be uniformly applied and spread without oversize particles, which most frequently are either crushed during rolling or fail to become embedded in the asphalt, and remain loose on the surface until broomed off or whipped off by the traffic.

An aggregate of approximately uniform size, with a maximum of voids is most desirable for penetration asphalt surface treatments, in order that a maximum amount of asphalt binder may be used without the aggregate becoming sufficiently embedded to result in a slick, bleeding surface. Also, the uniform size aggregates usually develop better interlocking qualities and provide lateral support to adjacent particles, thereby preventing displacement from traction and friction of high speed traffic.

The specifications should limit the amount of flat and elongated particles in the aggregate, and define what shall be considered flat and elongated particles. We are of the opinion that flat and elongated particles combined should not exceed ten percent of any gradation requirement. The following are suggested definitions for flat and elongated particles:

Flat particles are those particles with thickness of less than one half the average width of the particle.

Elongated particles are those particles with length greater than twice the other minimum dimension.

The following construction requirements and methods are recommended:

The temperatures of the air, aggregate and base or surface should be considered as controlling factors in determining suitable conditions for applying asphalt surface treatments, since each of these factors have much to do with the success of the application, in that the asphalt must remain in a tacky condition until covered and rolled, in order to obtain a satisfactory surface treatment.

The hard asphalts should be applied only when weather conditions are favorable, and when the air, aggregate and base or surface temperatures are each at or above 70° F.

Liquid cut back asphalts, road oils or emulsions may be used if weather conditons are favorable, and the air, aggregate and base or surface temperatures are each above 40° F.

The application temperatures of hard asphalts should be controlled within certain limits, depending on the viscosity of the asphalt. Asphalt should not be applied at a greater temperature than necessary, because the process of spraying asphalt at excess temperature reduces its penetration and ductility factors, and frequently results in gravity flow of the applied asphalt, on inclined surfaces.

The viscosity of asphalt varies greatly with changes in temperature, and the viscosities of various asphalts vary greatly at the same temperature.

We have observed that best results can be obtained by heating the asphalts to such temperatures as to produce viscosities within a range of 40 to 60, with a possible optimum of 50.

Immediately after the asphalt has been applied, it should be uniformly covered with aggregate and thoroughly rolled with flat wheel and pneumatic rollers. Any void spots in the aggregate cover material should be filled, and any excess of aggregate should be removed prior to the first rolling. No blading nor brooming of aggregate should be allowed prior to first rolling. Early the following morning, or when the asphalt has become chilled and hard, the aggregate should be lightly but thoroughly broomed with a drag broom and then after the asphalt has become warm and tacky, the surface should again be thoroughly rolled with both flat wheel and pneumatic rollers. Early the following morning, or when the asphalt has again become chilled and hard, the completed surface should be lightly broomed with a rotary broom sufficient to remove any excess aggregate that may be loose on

the surface, and rolled again as may be necessary.

In placing double asphalt surface treatments, it is necessary to apply a sufficient amount of asphalt to adequately retain each application of aggregate, but not an amount of asphalt that will fill the voids in the total compacted aggregates to an extent that will result in a slick, bleeding surface. It is our opinion that the average size particles of the No. 1 stone should control the resulting combined thickness of the surface. The No. 2 stone should be selected as to size, and placed in such amounts, dependent upon the thickness of and amount of voids in the compacted first course. In other words, the purpose of the second course of asphalt and aggregate should be to increase the density and improve the riding and water proofing qualities of the surface, and not to increase the thickness of the surface.

A coarse graded aggregate should be selected for the No. 1 stone; a fine graded aggregate of less than one half the average size of the No. 1 aggregate should be selected for the No. 2 aggregate.

The quantity of No. 1 aggregate should be determined the same as for a single surface treatment as previously suggested in this paper.

The quantity of No. 2 aggregate should be the maximum amount that can be added to the No. 1 aggregate in obtaining the maximum density or unit weight without increasing the volume.

This quantity can be determined in the laboratory by taking the unit weight of a given quantity of No. 1 aggregate, then adding and mixing quantities of the No. 2 aggregate until the maximum unit weight and density of the combined aggregates is obtained. The ratio in pounds of the No. 1 aggregate to the No. 2 aggregate, as reflected by laboratory tests, should be used as the ratio in pounds of No. 1 aggregate to No. 2 aggregate applied.

The next step is: From the unit weight and specific gravity of the combined aggregates the percent of voids should be determined, and the amount of asphalt required to obtain the desired embedded depth should be determined.

In applying the double asphalt surface treatment, the following is recommended:

The first application of asphalt should be one half of the asphalt required for the double surface treatment. The No. 1 aggregate should be uniformly applied, rolled and broomed as recommended for a single surface treatment.

The second application of asphalt and aggregate should not be applied until the first application has been thoroughly rolled and all excess, loose aggregate removed from the surface.

The second application of asphalt and aggregate should then be uniformly applied, rolled and broomed as recommended for a single surface treatment.

In conclusion, we are of the opinion that there should be closer control on gradation of aggregates and temperatures, and on rates of application of asphalts and aggregates, and as in the case of asphaltic concrete, there should be some uniformity in design methods for Asphalt Surface Treatments.

We do not presume that the design and test procedures, and application methods presented herein are the final answer. However, we do hope that some of the suggestions advanced in this paper will improve our work, and assist in arriving at the final answer.

Thank You.



Mr. Kearby and J. R. Mercer, Engineering Assistant.

TEMPERATURE-VISCOSITY CHART OIL ASPHALTS

O.A.-230

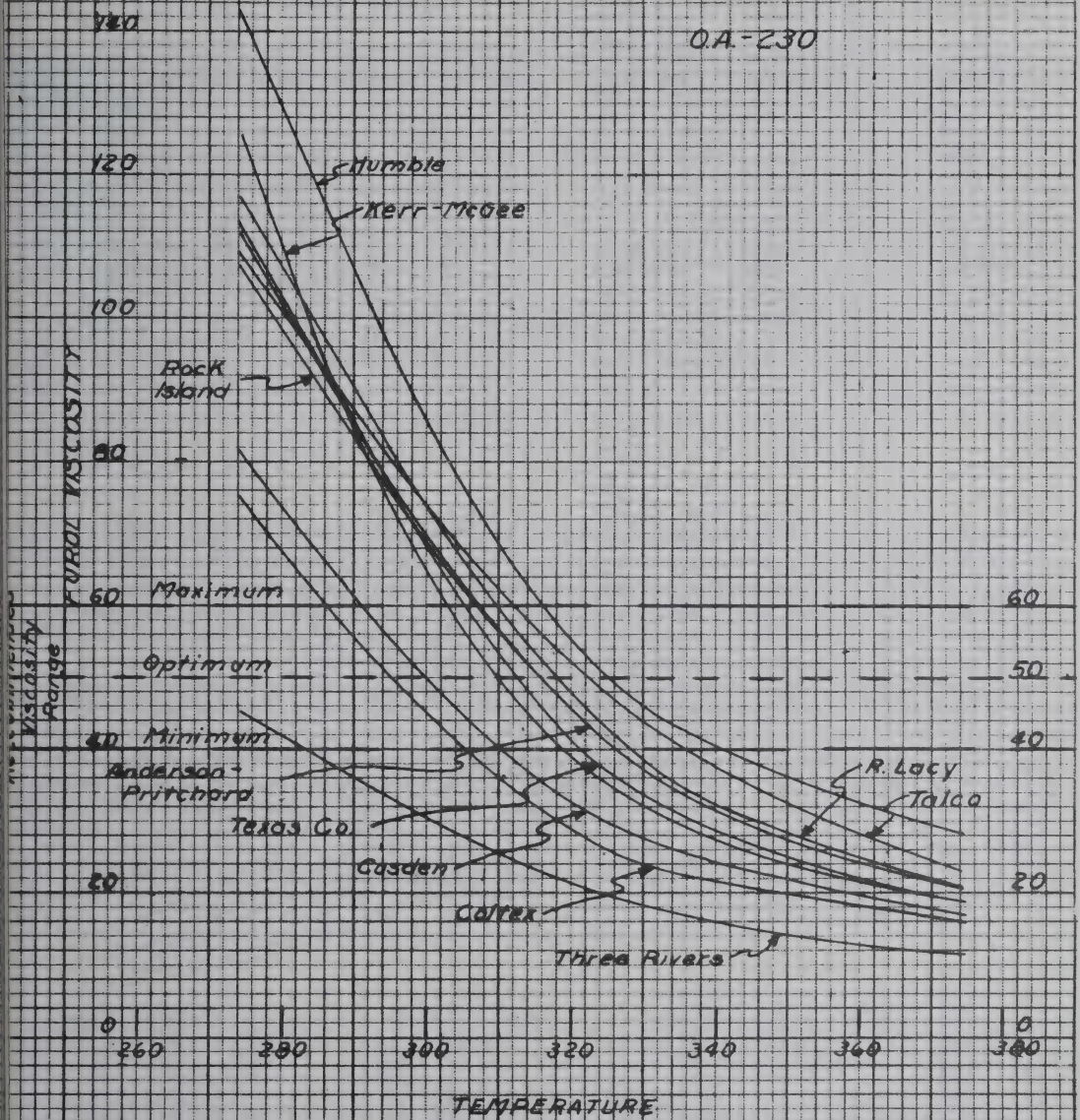


Fig. - VII A

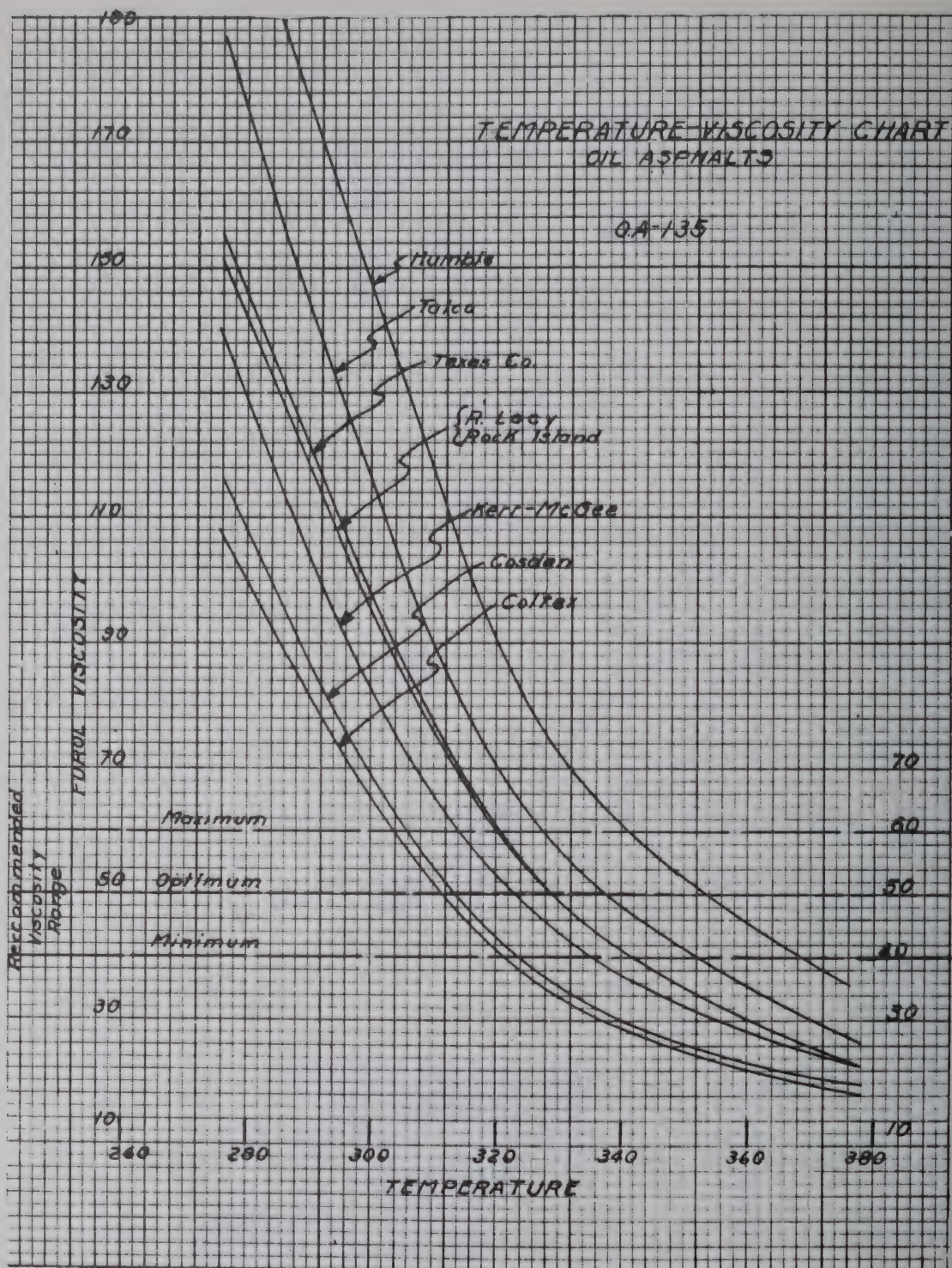
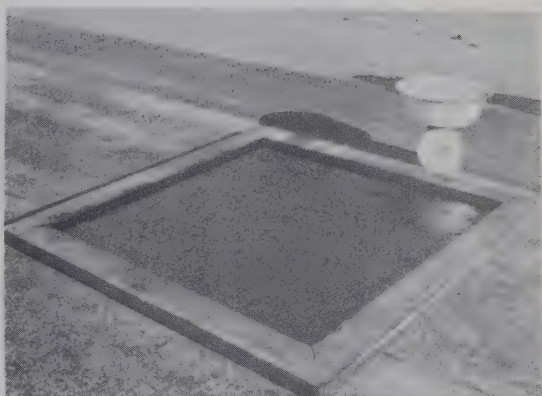


Fig. - VII B

SQUARE YARD TEST BOARD PROCEDURE ASPHALT SURFACE TREATMENT



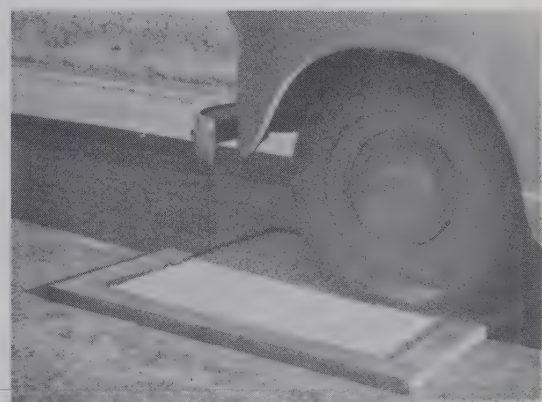
Square Yard Test Board in place, ready for asphalt application.



Square Yard Test Board with asphalt applied.



Square Yard Test Board with aggregate applied.



Square Yard Specimen receiving compaction.



Square Yard Test Board, Retention test.



Square Yard Specimen after compaction and retention test.

ASPHALT SURFACE TREATMENT TEXTURE ONE STONE COVER, CONTROLLED EMBEDDED



Asphalt, 0.25 gal. and No. 5 Crushed Stone.



Texture - Completed Seal Coat, 0.20 gal. asphalt and No. 7 Crushed Stone.



End of completed Seal Coat, 0.19 gal. asphalt and No. 7 Crushed Stone.



Texture of a Seal Coat after five years of heavy traffic.



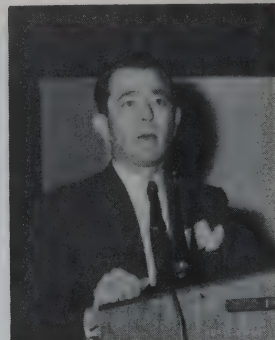
Completed Seal Coat, 0.20 gal. asphalt and No. 7 Crushed Stone.



Completed Seal Coat, 0.19 gal. asphalt and No. 7 Crushed Stone.



10:25 Construction of Underreamed Foundations
for Structures
W. C. Raby, Sr. Resident Engineer
Texas Highway Department
San Antonio, Texas



CONSTRUCTION OF UNDERREAMED FOUNDATIONS FOR STRUCTURES
By W.C. Raby

It has long been a desire of designers to add to the structural advantages of pile foundations, those of a spread footing without the excessive cost resulting from structural excavation and forming below the ground. The underreamed foundation provides this combination in this unique method of transmitting superstructure loads to a deep-lying stratum with a minimum of excavation and soil disturbance.

While we are particularly interested in the application of this foundation to highway structures, they are also extensively used for many other types of construction, particularly in buildings of all sizes.

This discussion will have particular reference to our experience with underreamed foundations in the San Antonio area, however, many of the problems presented in their construction are similar in any part of the State.

Before taking up the matter of construction it will be of interest to consider briefly the history of the development of this type of foundation, its economical aspects, and to touch on some features of design.

As a great industrial development took place in this country after 1900 with a resulting rise in land values, designers and builders were confronted with the problem of providing foundations for larger and heavier buildings on smaller areas. In the San Antonio area during this period, foundations for buildings consisted largely of two types of foundations, cast-in-place or "friction" piling and spread footings. Designers attempted to meet the challenge presented in creating foundations of a higher bearing capacity by enlarging on these two types merely by deepening the friction piling or enlarging the areas of spread footings. The crude methods available for drilling the holes for the friction piling were being strained beyond the

point of economic feasibility as it was required to reach greater and greater depths. As building areas were reduced, a mere enlargement of the bearing area for spread footings did not provide an answer and these had to be carried deeper to reach strata of higher bearing capacity, resulting in excessive costs brought on by structural excavation and forming below the ground. Driven piling were not adaptable here as they could not be driven without the use of drilled pilot holes carried to great depths.

To overcome these difficulties, it could be seen that a combination of the advantages of the two types of foundations would automatically eliminate the disadvantages of each type used separately. In 1925, Mr. Willard E. Simpson, Consulting Engineer in San Antonio, was successful in providing this combination in a residential foundation which consisted of holes drilled to a depth of about 18 feet, the bottoms of which were enlarged by hand methods by men working on the ground with long-handled tools. This worked very well in this case, as a satisfactory bearing material was found at this shallow depth, but this was about the limit to which this method could be applied. Experience had shown that foundations in this area should not be landed on clay at depths smaller than about 20 feet due to volumetric changes brought on by seasonal moisture changes in this formation. The problem of reaming-out at greater depths was met with the development by one of the local contractors of a crude reaming device attached to the end of the drill stem and operated with horse-drawn power. The first major building foundation undertaken using this method was in 1930 when the Jefferson High School building was constructed on underreamed shafts at a depth of 35 feet under the engineering supervision of Mr. Simpson. By this time, it was known that workmen could enter the drilled holes and enlarge the bottoms to a greater extent than was possible with the reaming device. Many difficulties yet had to be overcome, however, mainly those having to do with handling of seepage water at great depths and the achievement of greater speed in the drilling of the holes. These difficulties were met soon after 1930 with the development of power machinery adapted to accomplish the drilling and to set and remove steel casings used for sealing out water.

This development also presented the means of drilling to the depths necessary to reach the blue shale formation underlying the clay in the San Antonio area, thus allowing designers to make use of this highly desirable bearing material, having a bearing resistance many times greater than that of the clay.

New construction techniques then followed in rapid succession until at present this work has developed into an industry of major proportions in Texas, which although very highly specialized has proven to be very satisfactory in providing an answer to many of our foundation problems.

In considering the economy of this type of foundation, it is readily seen that the old spread footings can no longer compete with this method when working at the depths necessary in the San Antonio area. To illustrate the economy gained, a typical bridge foundation in the San Antonio area will be considered. Four types of foundations will be compared: 1. Underreamed Shafts, 2. Concrete Piling, 3. Steel Bearing Piling, and 4. Spread Footing. It is assumed that each of these footings will be landed on the same bearing material, the blue shale, for this is the usual case in all of the types considered. All types will be considered to reach to natural ground level from a depth of 30 feet. Using a 30" shaft with a $5\frac{1}{2}$ foot bell, and with an allowable bearing resistance of 5 tons/square foot, the underreamed shaft is good for 120 tons. Allowing 60 tons per pile will require 2 piling of either concrete or steel. The spread footing will require about $3\frac{1}{4}$ square feet. In the case of the piling, pilot holes will be necessary and a cap will have to be placed at the ground level to distribute the load from the super structure, and to provide the lateral stability necessary in a piling foundation. Using current prices in the San Antonio area, the cost of the underreamed foundation is \$490. Precast concrete piling will cost \$850. Steel H Beam Piling will cost \$765, and the Reinforced Concrete Spread Footing will cost \$1025. For smaller bridges used throughout the State, these differences in cost do not follow the same pattern except for spread footings at depth greater than about 17 feet.

The wider use of this type of foundation throughout the State, will result

in lower costs and this favorable cost comparison existing in the San Antonio area will be approached for the piling foundations. It is interesting to note here that our records show that the average 1951 costs of underreamed shafts on the San Antonio Urban Expressway are lower than the State-wide 1951 average by from 25 to 45 percent for shafts of 30" to 18" diameter, respectively. This, of course, is due to the greater volume of this work concentrated in one area. We are using this type of foundation for all types of structures including slab bridges, continuous I-beam spans, rigid frame undercrossing structures, plate girder bridges and retaining wall footings. It is not inferred here that piling and spread footings will be crowded out of the picture on our bridges throughout the State. The type of footing used should be chosen only after weighing the design requirements against cost. It is our opinion based on our experience with underreamed foundations that they are applicable to almost any set of soil conditions and should not be ruled out merely on the basis of poor soil conditions existing above the bearing material. The depths to which these footings can be carried depends on the length of boom on the drilling machine. The length of boom provided on a particular project is determined by job requirements. Additional depths after the boom limit is reached are secured by use of a "stinger" or splice in the drill stem. They can be used in any type of surface condition if means of supporting the drilling machine can be provided. The range of shaft diameter considered to be within practical limits of present equipment is from 18" to 60".

Since the beginning of construction on the San Antonio Urban Expressway Project in 1947, there has been completed or is under contract a total of 22,000 lineal feet of these shafts of the various sizes from 18" diameter to 48" diameter. Much of this work is completed and carrying traffic and we have noted no trouble to date attributable to the underreamed shaft construction. Other untold thousands of feet of this type of foundation are supporting successfully many buildings in this area. In fact, it is a rarity when any other type of foundation is used for major structures in this locality.

In the design of these foundations, proper consideration must be given the lateral forces transmitted from the super-structure and from stream flow. These shafts must be considered as columns acting in direct compression combined with bending. Sufficient size of shaft and percentage of reinforcing should be provided to carry this combination of stresses. The length of column should be taken to reach a depth below which it will have full lateral support from the adjacent soil. Thus it can be seen that the size of the shaft as well as the size of bell are dependent upon the capacity of the earth to resist the forces, either lateral or direct, to which they are subjected.

In discussing the construction techniques involved in this type of foundation, three important requirements are vital in the achievement of adequate foundations under all conditions encountered.

1. Trained crews operating under expert supervision and using adequate equipment.
2. Strict observance of the physical laws relating to fluid pressure, friction, and setting properties of concrete.
3. Professional engineering supervision supplemented by trained inspectors capable of exercising good engineering judgment.

Underreamed foundations should not be undertaken unless these three fundamentals are observed to a reasonably high degree. Cases of failure of these footings now on record are probably due to the lack of one or more of these requirements.

These three factors are brought into play more extensively when casings are used to seal off water and this particular construction procedure will be discussed in order to illustrate their importance.

It will be assumed that a 36" Diameter Shaft is to be drilled to a depth of 40 feet, the bell to be landed a minimum of one foot into the blue shale. The bell is to be 6 feet in diameter and 3'-10" high.

The exact position has been staked by the field party. The contractor drives 2 - 2x4 reference stakes braced in two directions at a sufficient distance from the hole to be clear of drilling operations, usually about 3 feet each side of the center of shafts. The referencing is done with a plumb bob and a wooden straightedge notched at the center and on each end at equal distances from the center. Grade nails are driven in the sides of each of the reference stakes, both nails being at the same elevation, and for convenience are set to an even foot. A 4" diameter pilot hole is dug by hand to a depth of about 18", just enough to guide the drilling bucket until it bites in.

Reinforcing steel has been delivered previously and steel tyers are engaged in making up the 30" diameter cage consisting of 18 - 1 $\frac{1}{4}$ " square longitudinal bars with 3/8" ϕ spiral hooping at 3" pitch. The length of steel used is based on that indicated by the test hole data shown on the plans with some additional steel on hand for splicing in case the shale is deeper than anticipated. The spiral hooping will embrace about 37 lineal feet of the main steel, as the bottom 3 feet is not hooped to eliminate obstructions to the flow of concrete in the bell. There are 150 turns of the spiral or 2700 intersections. It is not necessary to tie each intersection if a sufficient number of double or two-way ties are used interspersed with single ties staggered to give about 25 ties per lineal foot of cage. Every intersection for the top two feet of cage should be tied with double ties to prevent "racking" while being picked up. The number of ties used is not so important as the manner in which the tying is done.

A 1/2 C.Y. machine equipped with drilling table and a 70 foot boom is used for drilling. This machine could drill to a depth of about 55 feet with a single drill stem. A 4" diameter hole is drilled using a cylindrical bucket type drill equipped with cutting blades placed on the proper angle to force the excavated material into the cylinder. The dirt falls out of the bucket when the hinged bottom is unlatched. The 4" diameter hole is used because it is anticipated that water will be encountered and about 3"

this drilling operation to determine the plumbness of the hole by dropping a plumb bob to the bottom from the notched template previously mentioned. Changes in direction of drilling are made by manipulation of the boom.

Water is encountered at a depth of 30 feet, and a 37" Diameter casing made from 5/16" steel plate is set in the hole, centered at top and bottom, blocked at the top, and driven about 6" into the clay just below the water stratum. The driving is accomplished by raising and dropping the drill stem on a heavy round steel plate placed on top of the casing. After the driving is done and it is seen that water has been successfully sealed out, the casing is brought to the correct position again and the space between the casing and sides of the hole is filled with pea-gravel which serves to hold the casing in the correct position and will prevent caving of loose materials into this space. This pea-gravel is placed to an elevation just below the ground surface. It has been found that pea-gravel offers less frictional resistance in the casing pulling operation than any other material. Sand has been used, but it has a tendency to grab the casing and make pulling a great deal more difficult. The greatest benefits derived from filling this space between the casing and sides of the hole is the holding of the casing in its required position and the reduction in the amount of concrete necessary. Drilling is then resumed using a 36" diameter auger-type drill with pitched blades designed to carry the excavated material out. This material is deposited from the bit, merely by alternate reversals of direction of rotations of the drill stem in quick, jerking motions. The drill will cut from 2 - 3 feet of depth at a time.

Material having the appearance of blue shale is encountered at 40 feet, but the inspector on observing closely the material coming out sees that it is a soap-stone formation which is frequently found just above the shale and requests the drilling to continue. The blue shale is then encountered at 42 feet and the hole is drilled one foot deeper to provide good anchorage

in the hard material. Steps are taken to splice the steel to accommodate this 3 foot overrun in depth.

A reaming device is not used on this bell as the hole is large enough for a man to work in and this is considered to be the most economical method of belling.

In order to achieve proper sequence where a number of shafts are to be drilled, it is found to be advantageous to set casings on one day and drill to shale and begin belling on several holes early the following day in order that belling and concrete placing can all be completed in a single day. It is not wise to complete a hole if it cannot be poured during the same day. Maximum use of the air compressor is made by following this sequence.

For the belling operation, a four legged steel frame with pulley and hand-windlass is placed over the hole and a bell man is lowered to the bottom on a rope. Belling is accomplished by two men working together, one in the hole, the other on top bringing out the excavated material. The excavation is done with a small air hammer which is just a miniature of the ordinary air spade. A small portable generator is on hand to supply light at the bottom of the hole. This belling operation requires from $3\frac{1}{2}$ to 5 hours depending on the skill and energy of the individual men. About 1 C.Y. of material will have been removed when completed. The inspector now enters the hole to check the bottom before steel is placed. He takes with him a small geologist's pick and a rule for making measurements. He is careful to watch for loose material or for material which was once disturbed and then re-compacted by the bellman walking around on the bottom. The inspector then comes out, makes notes in his diary as to the condition of the hole and if no corrections are to be made, he measures the exact depth of the shaft for payment, making use of the grade nails on the reference stakes. The required amount of concrete is ordered from the Transit-Mix Company to be delivered at a particular time. The cage of reinforcing steel is lowered into the hole and is supported by 6 x 6 x 8" concrete blocks with holes in which the bars are held and with

wires tied securely to the steel. Spacer blocks are tied to the cage at intervals of about 10 feet to provide a uniform clearance between the casing and the reinforcing steel.

No concrete is placed until the amount required to complete a shaft is in sight. When using casing, all efforts are made to reduce delays in pouring to a minimum. The loss of an expensive casing may be the result of lost time during the pouring operation. The slump of the concrete is of particular importance in this work and we arrange to have a laboratory inspector at the concrete plant to insure the delivery of concrete having the desired slump. It is imperative that the concrete be of such consistency as will provide near equivalent fluid pressure to that of water on the outside of the casing. Failure to observe this simple fact is probably responsible for some of the bad results on record in this type of work. We have been keeping a close check on concrete strength by making numerous test beams, and our average flexural strength is above 700 lbs/sq. in. for the 5 Sack per C.Y. Concrete which is specified. If it is felt that strength will suffer appreciably by using 5" - 6" slump concrete, the cement content should be raised to 6 sacks per C.Y.

On our highway work we have been requiring the vibration of the bell concrete to insure full contact over the entire bearing area and to completely fill out the bell. This usually requires not more than about five minutes.

After vibrating the bell concrete, the shaft is filled to an elevation well above the level of the water on the outside of the casing, the differential being adequate to overcome the friction of the concrete against the casing and the resistance of the reinforcing steel to the flow of concrete to the outside diameter of the shaft. It is well to take measures to reduce the frictional resistance between the casing and the concrete by oiling the inside of the casing before each use. A casing pulling machine is moved into position and a cable is attached and pulling is begun. This operation is watched very closely to observe any rise in the reinforcing steel with

the casing. If this happens and it is found to be impossible to shake the casing loose by vibration or by sledge hammer blows on the casing, the pulling operation should be discontinued after time has elapsed approaching that of the time of initial set of the cement. In our experience, the chance of this happening is very slight. We have had three occasions out of 600 cased shafts where the steel rose with the casing, and in these the concrete was still plastic, was vibrated to the bottom of the casing, and the casing left in place.

After the casing has been partially removed and raised several feet, additional concrete is placed inside to insure it reaching to the approximate required grade. The casing is then entirely removed. The shaft concrete is then vibrated with the mechanical vibrator to the depth of the original position of the bottom of the casing. This final vibrating must be done with care and to a moderate amount to avoid mixing of the water on the outside of the shaft with the fresh concrete.

The reinforcing steel is checked for position and a steel form is set in position to form the top 6" to 12" of the shaft for subsequent connection to the column above. This form is centered exactly and filled to the required grade. This forming is not necessary on units where column continuity is not called for, such as in retaining walls, abutments, and solid pier footings. The top is floated with a wood float and when sufficient time has elapsed to allow it, moist sand is placed on top for curing. Laitence is removed, preferably just before final set of the cement takes place. It is considered of utmost importance in this work that, after placing of concrete is begun, there be no delays in the completion of the shaft. This is where expert supervision and experienced inspection working together pay off in satisfying results, both to the contractor and to the State. I have described here a case where conditions are considered to be normal. Actually, it is the abnormal conditions which frequently confront us that call for ingenuity on the part of those doing the work. We should not gamble on a bridge foundation. We should be in a position to know that unusual conditions or difficulties encountered

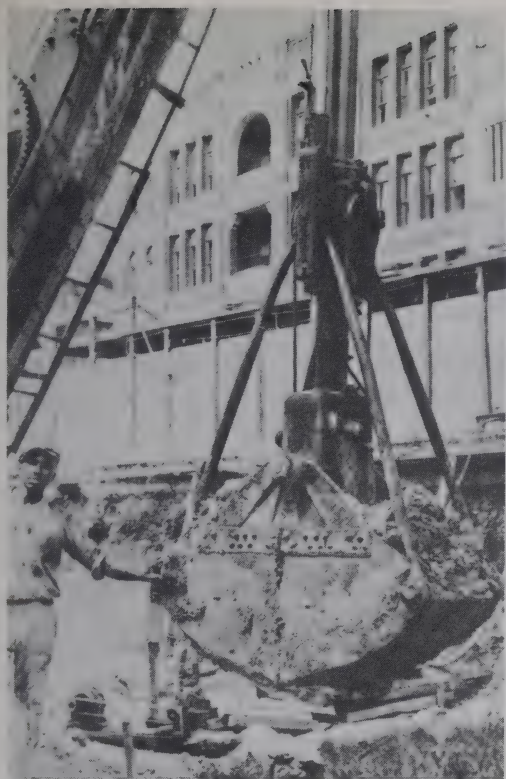
in this work are met satisfactorily and that the results are not less than those expected by the designer in creating the plans. We can be in this position by adhering strictly to the three basic requirements previously mentioned.

Much has been said in recent months pro and con on the matter of removing the casings from underreamed shafts. It is felt by some that the casing should be left in place to obviate the intrusion of mud and water into the fresh concrete. There are cases on record where this has happened. There are also cases where the shaft concrete was pulled completely apart leaving nothing but the reinforcing steel to carry the load. A detail study of these cases will show that at some time during the construction, the simple fundamentals previously described here were missing.

The question has been raised as to whether or not to vibrate the shaft concrete with mechanical vibrators. It is believed that this serves a good purpose if done with care and in a moderate amount after the casing is removed.

The matter of segregation is also of some concern. Our specifications require that some form of tremie pipe be used to place the concrete to eliminate segregation and coating of the reinforcing steel. It is our opinion that this requirement should not be included in the specification. We have seen segregation take place to just as large extent when using tremie pipes as when they are not used. We do not believe the results of their use justifies the loss of time during this critical stage of the operation. The matter of steel coating is not important as the pouring operation requires so little time that the coating which does take place does not dry sufficiently to be objectionable.

In conclusion I wish to express the hope that this report has served to dispel any doubts that may have been held concerning this type of construction and that you will consider the underreamed foundations as another very effective tool at our disposal in providing adequate as well as economical foundations for our structures.



Rig used for excavating open holes for spread footings before development of Drilled Underreamed Foundations.



Horse-drawn rig used in early 1920's for drilling holes for cast-in-place concrete piling. Later used to drill underreamed foundations before power equipment was developed.



Modern reaming device used in belling operation on underreamed foundations. (Open Position)



Reaming device shown in No. 3 showing closed position.

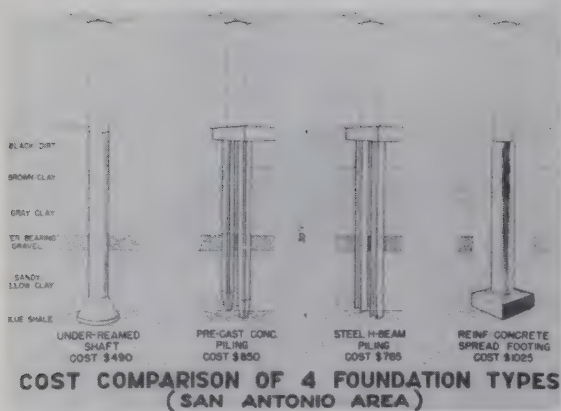
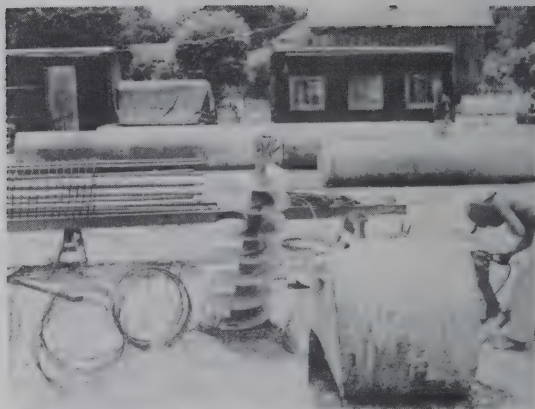
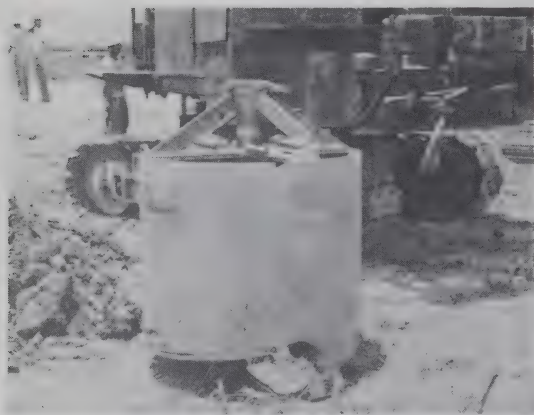


Chart showing cost comparison of four foundation types in the San Antonio area.



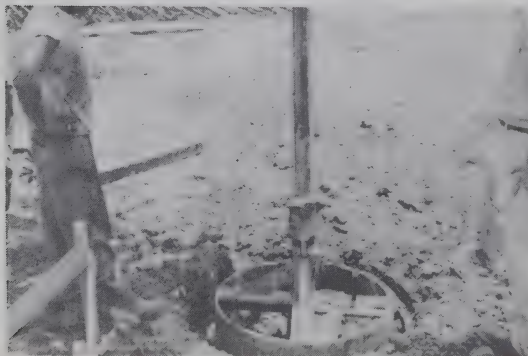
Two types of drills used when casing a hole. On right, the bucket-type drill, on left, the auger-type drill.



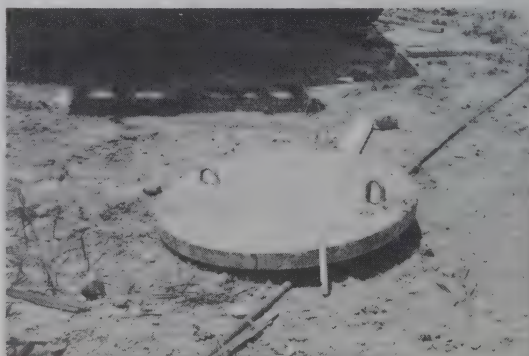
Bucket-type drill used to drill over-size hole to accommodate a casing.



Auger-type drill.

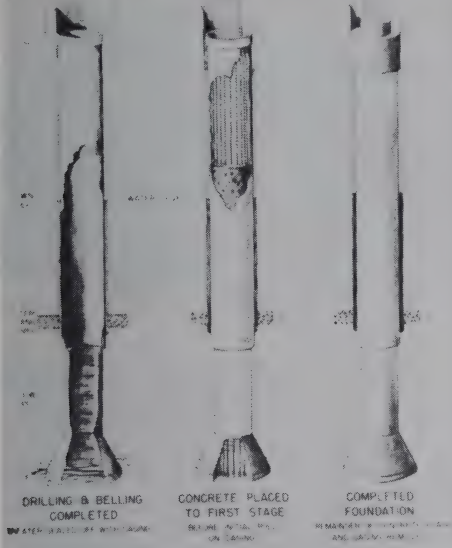


Bucket-type drill in operation.



Steel plate used in driving casing to seal off water.

CONSTRUCTION STAGES OF UNDERREAMED FOUNDATIONS

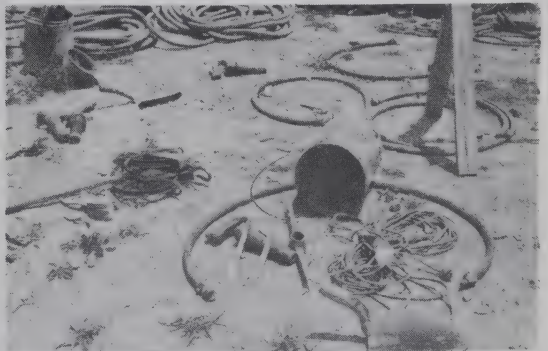


Construction stages of underreamed foundations.

Modern rig equipped with drilling table and 70 foot boom used in drilling, setting reinforcing steel for underreamed foundations.



Frames used during belling operations.



Tools used by bellman in underreaming.



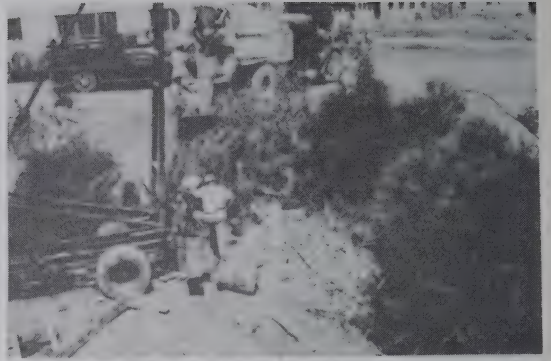
Close-up of air hammer used in belling operation.



Cased hole finished and ready for final inspection before setting reinforcing steel and pouring concrete.



Inspector preparing to enter hole for final inspection.



Casing pulling rig in position for removing casing.



Cased shaft in place where casing was left in after attempts to pull it failed.



Final checking of reinforcing steel after concrete is placed and casing removed.



Reinforcing steel being lowered into hole.



Exposed underreamed shafts which had been constructed to support property adjacent to a large basement for a San Antonio building.



Close-up of one of the exposed shafts in previous picture.



Underreamed shaft removed after 10 years in use.



Close-up of bell on shaft shown in previous picture.



Slab bridge supported by underreamed foundations.



Partially completed continuous span I-beam bridge showing piers supported by underreamed foundations.



Completed I-beam bridge supported by underreamed foundations.



Rigid frame bridge supported by underreamed foundations.



Plate girder bridge carrying railway traffic, which is supported by 48 inch diameter underreamed foundations.



Completed expressway structures which are supported by underreamed foundations. Many of the buildings in the background are supported by this type of foundation.

10:45 Discussion
Verne Watson, Contractor
Fort Worth, Texas



DISCUSSION
OF
SPEECH BY W.C. RABY

CONSTRUCTION OF UNDERREAMED FOUNDATIONS FOR STRUCTURES

Gentlemen:

Mr. Raby has just finished a very good description of drilling conditions in San Antonio. I think that the contractors or anyone attempting to drill in this area under those conditions would do well to follow his instructions. If this is done, they will have a perfect job.

There are very few comments that I can make other than that it is an excellent report. What few criticisms I have would be so minor that they would be hardly worth mentioning.

You know, a drilling contractor has a similar position to this situation. A couple of fellows were making a trip on a train. The train finally pulled out and they were on the back platform. They noticed a little dog chasing them, running as fast as he could. Bill turned to John and said, "Do you think he'll catch the train?" John said, "I don't know; but what will he do with it if he does?" And so with the contractors. Sometimes we get these plans and wonder what the hell we're going to do with them. And in a few cases some of the fellows have taken on jobs and didn't understand just exactly what was to be done. They have had a few interruptions and costly delays because of improper drilling techniques or maybe improper equipment and a lot of other things probably could enter into it. And in a few cases they have had poor results. But haven't we all had it in every type of foundation at various times.

The method of drilling, as Mr. Raby pointed out, in his particular area has to do with an impervious clay. It will stand open for days without any trouble. Occasionally they run into a water stratum down at various elevations from 20 to 30 feet, but from there on down there is no casing required in the hole at all. The method of staking off is necessary, I think, for references, because when you destroy the center stake that the engineer has placed there, you must have an alternate to go by to keep the center of your hole. However, these conditions do not exist in all parts of the state and we have played the field and have taken the jobs where they have shown problems requiring a lot of experience in meeting conditions and coping with the different situations.

For instance, Mr. Raby has explained a technique where complexities are at a minimum. It would not apply in a case where you have started drilling and the ground begins to fall out from under you at the beginning, where water is maybe 6 or 8 inches below the top surface of the ground, where you have a fluid condition of the sand, where if you removed any of the soils, the earth around you would cave in. Necessarily that means a different technique of drilling. That challenge is being met every day by a great number of drillers in the business. We have had that situation come up not once but several times recently near the San Antonio area. You don't have to move far from any one area to get a soil change. In this particular instance, only 60 or 65 miles from San Antonio, we found a fluid condition from the top of the ground down to about 35 or 40 feet. There we encountered an impervious clay in which we could seal water off and bell below. Now, we hold these walls up; we don't have any excessive caves. Concrete is probably 5% more than what you would necessarily need by having the casing in the hole anyway. And these conditions cause a lot of reluctance in some localities with engineers who are not familiar with them, or who have perhaps seen someone try to drill in that particular soil and say, "Well, you just can't drill a hole there." That's erroneous; it can be drilled. Any particular area does not mean that you have to go to some other type of foundation when you have this particular method of installing a foundation available. There is another thing that is causing some concern among engineers. I think the State Highway Department has had it proved to them several times and I don't think the designing engineers in the Bridge Department are too much concerned about it because they have seen it done, where they have a lot of overlying deep sands, fluid sands and then right into rock.

There is another drilling technique required when you have to seal water off on top of this rock. By the method that Mr. Raby gave this is practically an impossibility. You have to enter into that rock. It cannot be done by driving. If the casing material would stand the driving the rock would be shattered and you wouldn't get a seal. So there has to be another method. It is being done every day. There have been found some design requirements where a key of some few feet is necessary to transmit intense lateral pressures to a solid base material. This can be done by a coring method that has been developed to an economic degree. There are now bridge structures in the state with 42" shafts extended into solid rock. Certainly no rock excavation is a cheap operation - but where this is a design requirement it is an economical method.

I notice that prices on drilled shafts have shown an increase in the State Highway report for 1951. There is a very logical explanation for this. There has been a very small amount of footage drilled outside the San Antonio area, in other parts of the state, in comparison with the number of feet of piling. For instance, we had one job that had, I think 90 feet of drilled shaft in it, and we had to go over 100 miles to do that, and necessarily the price would be higher. It was necessary to charge all our moving in and moving out against a small amount of footage.

And about design: I would like to ask a question of the engineers in this group - designing engineers and structural engineers. I notice Mr. Raby said in his talk that they would have to put in two concrete or steel piles at the same elevation and in the same type of soil that they would put in a 30" shaft with a belled footing. This is something I want to know. Why you arrive at your conclusion that two, especially H piling, will carry the same amount in the same type of soil as a 5'6" bell. Now you can imagine a 30" shaft drilled and belled. You can see a cross section in your mind and you can see on either side that, if the hole was open and a couple of piling driven down to that same elevation and allowed to set on the bottom of the bell, the area they would cover would be so small in comparison to the big spread of that footing there seems to be something wrong. I am just wondering what it is, and if it can be logically explained why that type of reasoning is followed.

I am disturbed about another thing. Fellows, this is, I think, a little bit of constructive criticism. I have had a case come up where the plans had an alternate. They had, on their base bid, a 10" H piling driven 22' down to rock, and they had three of those. There were two 30" diameter shafts with nearly as much reinforcing in them as the H-piling weighted, plus a 7' bell on top of this rock. Now - is that competitive? I am wondering why that particular thing has taken place.

We have seen some improvements made. Where you are going into base material such as shale, rock, limestone, they are gradually leaving off the bells in that particular area. They are making us go into the rock deeper, but they are making some improvements. I am not saying anything against the belled shaft, because I believe in belled shafts. I think they are proper; I think they are all right. But there is a place for bells as there is for spread footings, and that place is not where you go down to rock and you have an overlying surface there, an area of soil and perhaps 30 to 40 feet of overburden, and you land on solid rock. When you go into that rock with a shaft just why would

it require a big belled hole? Now, as I say, they are making some changes on that and we are only too glad to see them. Some of the cost is being cut.

I think that we are not taking advantage of the carrying capacity of some of these piers. I am not trying to say that your piers are too large. It may be that you need them. It may be, though, that you are not using the full capacity of these piers.

We recently made a test on an 18" pier. It is on record in your State Highway bridge department. That pier, with not a sign of a bell on it took 100 tons within ten days after being poured under water, under a fluid condition, only 22' in the ground, and showed no deflection whatsoever. One hundred tons on a straight 18" shaft, 22' deep. No deflection. That is an impressive figure. There was no failure - we do not know what it could have held. We reached the limit of the jack.

We made one in Houston under the direction of Fred Harris and the pier, a straight 20" shaft, went to 186 tons there was no failure point reached. In Houston, where a relatively firm bearing material is relative only to Houston. I am just wondering if sometimes we are not being a little overcautious because of unknown factors that are involved in drill shafts, that there has not been enough study made on it.

To summarize my views in improving the practical usefulness of drilled shaft footings:

It is necessary that job inspectors be educated in the proper procedures that prevent failures in installation - control of concrete aggregates and moisture content. From past experience we have found that there is justification for requiring the use of an air entraining agent, such as Darex, so moisture content can be kept to a minimum. Any excess water in concrete placed rapidly to a depth of 20' or over will literally be pressed out of the mixture, and is relieved only by "bleeding" up through the column, which results in a washing of cement from the aggregate in the channels it follows. This causes a loss in lubrication from the cement as well as a weakening of the concrete mixture. The lubrication lost was that needed to insure the withdrawal of the temporary casing without disturbing the reinforcing steel. This is an effect that will probably not occur where the shaft is extended to any great depth below the cased area, for the obvious reason of having the steel anchored below the casing.

Following this, there is a conclusion that no vibrating is necessary because of the rapidity of concrete placement and the effect of vibration that occurs from the placement itself. Further, it will be found that the Hydrostatic pressure of a rapidly placed

cause some impregnation of the soils themselves. If any vibration should be necessary it will only be the top 10' area and can be done after all concrete placement is completed and the casing removed.

There are a great many details in any construction operation and time will not permit the coverage of all these. So - I shall conclude in saying that only when drilled shafts are used extensively and are competitive in design will their full economic advantages be utilized.

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11:00 Progress Report on the Correlation of the
Static Load Capacity of a Pile with the
Shearing Strength of the Surrounding Soil
A. C. Kyser, Assistant District Engineer
Texas Highway Department
Houston, Texas



PROGRESS REPORT ON THE CORRELATION OF THE STATIC LOAD CAPACITY OF A
PILE WITH THE SHEARING STRENGTH OF THE SURROUNDING SOIL

By
A. C. Kyser, Ass't. District Engineer
District 12

During the past year, the Texas State Highway Department spent approximately \$10,200,000 for bridges including design and construction, but during the same period only about \$75,000 was spent for soils exploration at the bridge sites. This means that less than 1.0% of the money was spent to determine the type of formations on which these bridges are resting. It necessarily follows that information on foundations is essential before any economical and practical design of a bridge foundation can be made.

In the Gulf Coast area a medium or a stiff clay is considered good foundation material. In most cases, we are depending upon skin friction of the piling to support the piers. In this discussion, the term skin friction is referred to as the resistance to settlement of a pile created by the friction between the sides of a pile and the adjacent soil. In cohesive materials the value of the soil immediately below the bottom or tip of the pile is low and of questionable value, and is disregarded entirely in our calculations.

In the absence of complete soil information and a better method of determining the bearing value of piling, the engineering news formula or hammer formula has been used for the determination of the capacity of foundation piling. This formula attempts to correlate the weight and drop of the hammer, and the penetration produced by the blow to the safe bearing value of the piling.

Suppose we take a typical pile driving operation. At the start, let us say, the indicated capacity is fifteen tons. As the pile is driven, the computed bearing may increase to say twenty-five tons at a lower depth. At still lower depth, the computed bearing may drop back to below the original fifteen tons. It is obvious that we are measuring point resistance. We are getting point resistance and very little skin friction. If we were actually getting

a truly computed frictional resistance, it would be theoretically and practically impossible for our computed bearing to drop back from a previously computed higher bearing in an upper strata. Consequently, what we are doing is using a partially empirical equation which is based on the energy necessary to drive a pile through earth at a comparatively rapid rate. Also involved is point bearing, weight of the hammer, distance of the hammer fall, rate of speed at which the hammer is allowed to fall, and many other intangibles. During the driving operation, the skin friction is almost negligible since skin resistance has had no time to develop. From all of this, we attempt to determine the frictional resistance of the pile to the surrounding earth.

In short, we are measuring one thing and attempting to apply it to something else. It has been coincidental when the hammer formula checked the actual bearing capacity of the pile. For example, in a series of tests, using timber, concrete, and steel piling, it was found that in cohesive soils, the computed loads by using the hammer formulas to the actual ultimate loads as determined by load test varied from a minimum of 26% to a maximum of 630%.¹

The favorite argument for the use of the hammer formula is "We have been determining the load bearing capacity of piles for years and look how they have held up." This statement is partially true but only tells half the story. I wonder how much money has been spent in the past years making piles considerably longer than required to support the load with a reasonable factor of safety? Then there is another phase to the story. When a piling is driven under certain conditions, the bearing capacity appears to be greater than it really is. For example, driving piling a short distance into a stiff clay. Another fallacy of the hammer formula is that it does not give a true indication of the bearing capacity of a pile cluster.

Considerable time is lost and construction costs are increased because the specifications require the contractor to drive test piling, wait for graphs and other data to be prepared and interpreted, and the approved lengths agreed upon between the district, the Austin office, and the Bureau of Public Roads.

¹Proceedings - ASCE, Sept. 1941, p. 1393

If pile lengths can be determined by other means, a considerable saving in structure cost will be realized. Regarding the failure of structures which have been constructed in accordance with the usual engineering practice, the following example is a case in point. Auger and wash borings were taken, test piling were driven and the pile lengths were selected by the resident engineer, approved by the district office and the Austin Office. The piling were driven and adequate bearing obtained as computed by the Engineering News Formula. This structure was completed the latter part of last year and before the road was opened to thru traffic, one abutment bent had settled six inches.

The piling in this bent are thirty-four feet long. After the failure was observed, the site was drilled with one of the large rigs which work under the supervision of the Bridge Division and you can see as illustrated in Chart No. 1 the conditions that were encountered.

The piling developed a fair amount of resistance as they were driven through the new embankment approximately sixteen feet in thickness. The piling reached the design bearing when they were driven approximately one foot into the sand strata below. From the driving records there was no indication that trouble could be expected. A closer examination of this site shows that there is undoubtedly consolidation of the soft clay beneath the fill. The piles not only carry the weight of the bridge but also carry a portion of the weight of the embankment. The shear value of the soft clay layer is too small to measure. It is evident therefore that the pile is supported by the one foot of penetration into this sand layer. The foot of depth is insufficient to develop any large amount of skin friction. It appears, therefore, that the piles are essentially point bearing, and the actual condition indicates that to develop the design load, the computed bearing would be in excess of 30 tons per square foot on the sand layer. No wonder there is an appreciable settlement, even though this settlement occurred before heavy loads were using this bridge. It can be expected that there will be additional settlement in the future and expensive remedies will have to be undertaken.

Good engineering principles have forced us to seek other means and methods for computing the bearing values of friction piles. It has been observed that when a pile fails either by overloading or pulling, the soil around the pile

is sheared and a film of soil adheres to the sides of the pile. It therefore follows that the design criterion of most soils should be based on their shear value. Coulomb determined that the shear value of a soil is equal to $C + r \tan \phi$ where; C is the cohesion of the soil, r is the weight of the overburden and ϕ is the angle of internal friction. We need only to determine the shear value of the soil through an area equal to the outside surface area of the pile to determine load bearing capacity.

In District 12, the equation is used in the form: $P = (cAh \tan \phi)a$, where; P is the load bearing capacity of the pile, c is the cohesion of the soil, W is the weight of the overburden as determined from density tests (in most cases this is the submerged weight of the soil), h is the average depth of the layer below the ground surface, ϕ is the angle of internal friction, and "a" is the effective surface area of the pile for the particular soil strata being calculated.

A factor of primary importance in this method is the securing of undisturbed samples of soil at the proposed site of the bridge structure. Recent developments in the technique of soil sampling have enabled us to obtain undisturbed samples to a depth in excess of 100 feet in most cohesive soils. In fact, we have been able to obtain an undisturbed sample 4 inches in diameter and 8 inches in length of any soil that will stand under its own weight. Thus we have been able to compute the shear value of most of the soils encountered.

The Highway Department has several drilling rigs in use. The better of which is mounted on a tandem drive Ford F-7. This unit comes equipped with tracks that can be quickly attached or detached to facilitate mobility. This particular rig is a Failing 750 which means that this unit can core down to 750 feet. However, most of the work is confined to a maximum depth of around 100 feet.

Several features of this rig are the high pressure mud pump, automatic pulldown, hydraulic gauge and a rotary type drillhead or retractable table. A water truck is used in conjunction with the drilling rig. The mud pump serves a dual purpose in that it is used to help "clean out" a hole, mud the sides of the hole to minimize cave-ins, and extrude soil from the clean out barrel.

or constant rate at which the core barrel is pushed into the ground. The smooth operation of this device eliminates jerking or erratic movement of the core barrel in its downward progress, thus assuring the utmost care of the soil samples in the barrel. A hydraulic gauge attached to the automatic pulldown assists the operator to determine the type of formation he is in and also helps him determine when he is passing out of one formation into another.

The drillhead is a rotary type with hydraulic feed, 30 inch stroke, and high speed return. The drillhead retracts hydraulically to permit addition of drill pipe or casing.

In the past, samples were brought into the District Laboratory for testing. The transportation of these samples from the coring site to the district laboratory has been a difficult and tedious task because of chance of injury or damage due to moisture loss.

The tests are now being conducted in a field laboratory which operates from a pickup truck. A transmatic cell is used for confined compression tests. Nitrogen is used to furnish the required lateral pressure.

On large projects it is often desirable to run liquid limit, plastic limit and percent of moisture tests on the various strata in order to assist in properly grouping the materials and interpreting the triaxial test results. These Atterberg limit tests also serve to predetermine the consolidation characteristics. Standard penetrometer tests are used to assist the grouping of the soils. Sometimes on shallow explorations, several holes are drilled close together so as to make the identification of the samples more positive. Every undisturbed sample is tested.

Field testing can be done at about 80% of the cost of laboratory testing. It also eliminates any chance of having to retake samples because of damage in transporting or preserving them.

The stress-strain curves for the material are drawn and by use of the classification data, the Mohr circles are grouped and plotted. From the Mohr Circle is determined the value "c" or the cohesion of the material and ϕ which is the angle of internal friction. From this information, the estimated bearing capacity of the pile is calculated as shown in Chart No. 2. The available strength of each layer and the accumulated bearing value is plotted as shown

on Chart No. 4.

Load tests are being performed whenever possible to correlate the soil test data with the load carrying capacity of the pile. After the initial skepticism has been overcome in regards to this method, load test will only have to be employed where unusual soil conditions are encountered.

The proposed method of carefully analyzing the earth foundation material beneath the structure has many advantages; some of the more obvious ones are as follows:

- a. Explore the possibility of various types of foundations such as piling, drilled shafts with or without under-reamed footings and cast-in-place piles.
- b. More accurate determination of the piling lengths required to carry the designed load. This will result in saving of considerable amount of piling in many cases and prevent under design in others.
- c. Saving in construction time and costs by the elimination of the test piles. In many cases, job overruns will be eliminated.
- d. Reduce the dangers of settlement due to consolidation of lower strata. The tests will furnish data upon which to eliminate possible trouble spots.
- e. The true value of a pile cluster can be more accurately determined.
- f. Secure the necessary data for determination of maximum fill height.
- g. A uniform method for evaluating sub-soil data and eliminating the personal equation. It will furnish a basis upon which the results can be compared.

The general procedure in the past has been to conduct a load test on a test pile if it failed to show the design bearing as determined by the hammer formula. Generally the test was conducted only to the extent of proving that the pile was adequate for the design load. Only in a few cases have these tests been carried out far enough to determine the ultimate capacity of the pile. With a little more time, patience, and expense, a correlation of the soil test data could be secured which would be invaluable for future reference.

We have complete information on only three projects. The Camp Wallace

Road grade separation project is located on the new highway between Houston and Galveston. Good undisturbed cores of silty clay soil were taken and the required lengths of piles were estimated. When the contractor drove the test pile he secured a bearing of only 17 tons against 28 tons required by the specifications. To have lengthened the piling as required to secure the bearing by the hammer formula would have cost over \$9,000. By the results of confined compression tests on undisturbed samples, we reasoned that the load capacity of the piles of the length driven was 108 tons. The pile was loaded and the capacity found to be 115 tons.

Another load test was made on the San Bernard River Bridge, State Highway 60. Undisturbed samples were not taken at this location because the soil encountered was cohesionless sand and would not stay in the core barrel. The test pile as driven showed a safe bearing capacity by the hammer formula of 22 tons. The computed capacity for the pile based on the angle of friction as determined by the slope plain method on a disturbed sample was 100.8 tons. The capacity as determined by a load test for 1/4 inch net settlement was 80 tons.

A 25' steel H pile weighing 42 pounds per foot was driven to a depth of approximately 20' below the ground surface for a research project at Camp Hubbard. This work was performed under the supervision of the Bridge Division and was driven for the purpose of comparing soil test data with the dynamic resistance formulae. The results of the various methods of calculating the capacity of the pile as compared to the actual load depths are shown.

The summation of these results is shown graphically in Chart No. 5.

Conclusions:

Dynamic formulas are of questionable value in determining the safe bearing value of friction piles because they measure point resistance when the skin friction is low and after the pile is allowed to set, it is the skin friction that develops around the pile that is called on to carry the load.

The confined compression tests on undisturbed samples furnish us a direct approach to the problem in that the skin friction on the soil has a direct relationship with the shear value of the soil. Even allowing for the inaccuracies that may exist in the sampling and testing, it furnishes us a far

more accurate tool than ever before used.

This method is not new. It is being used by a number of the leading foundation experts throughout the country.

The results obtained so far in the coastal regions of the state point to a distinct possibility that pile load capacities can and should be based on the strength test of the supporting soils rather than an empirical dynamic formulae.

Acknowledgement:

I wish at this time to thank the various ones who have so generously assisted me in the preparation of this paper and have made an outstanding contribution to the cause; particularly Mr. Fred Harris of the Houston Urban Expressways, Professor Spencer Buchanan of Texas A & M College; Mr. Mack Shelby of the Bridge Division, and Mr. Frank Mitchell, Laboratory Engineer, District No. 12.

TYPICAL FOUNDATION PROBLEM

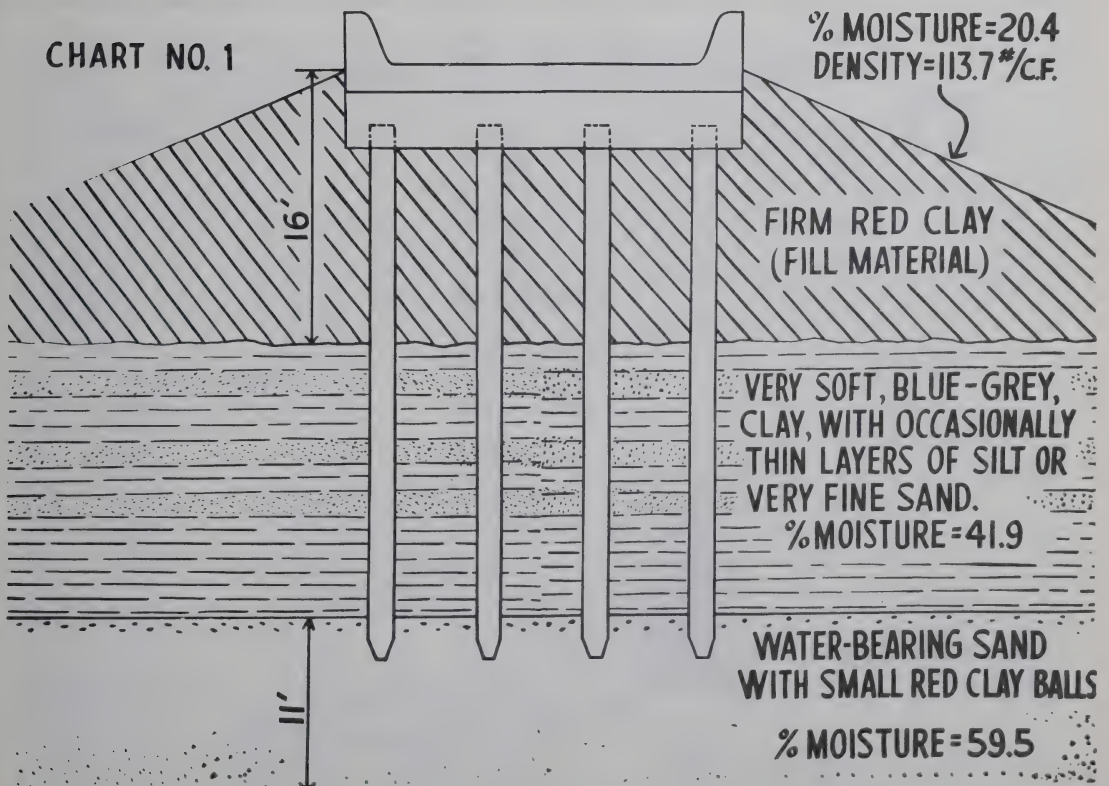
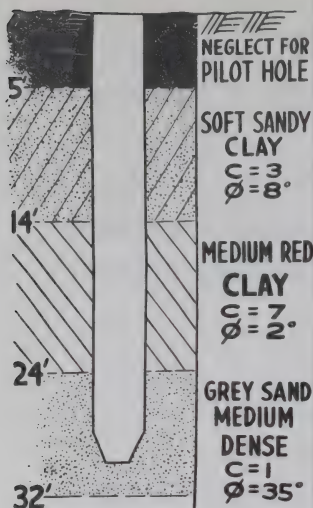


CHART NO. 2

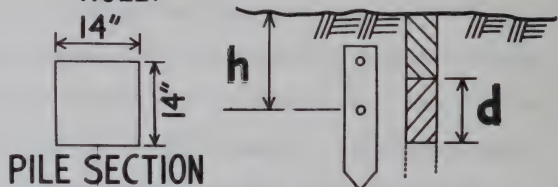


ESTIMATED BEARING CAPACITY OF PILES CALCULATED BY USE OF COULOMBS THEORY-T.H.D.

DEPTH IN FT.	d FT.	h FT.	W #/FT. ²	Ø DEG.	TAN. Ø	C #/FT. ²	Wh.TAN. #/FT. ²	C+Wh. TAN. Ø #/FT. ²	WRK. STRESS IN SF:2.0	BEARING IN LBS.	TONS	ACCUMULATED BEARING IN TONS
5-14	9	9.5	48	8	0.141	432	64	496	248	10423	5	5
14-24	10	19	52	2	0.035	1008	35	1043	512	23900	12	17
24-32	8	28	58	35	0.700	144	1137	1281	640	23900	12	29

$$FORMULA=P=(C+Wh \text{ TAN } \phi)a$$

REMARKS: 1. CALCULATIONS BASED ON
SUBMERGED DENSITY OF SOIL.
2. ELIMINATE 1ST. 5' FOR PILOT
HOLE.



d (depth of layer); h (depth to centroid of Strata); c (cohesion of soil = c from
TATx144); Ø (angle of internal friction); W (weight of soil in strata which is
usually the submerged weight of the soil = wet weight of soil - 62.4); a (sur-
face area or perimeter of piling x depth of strata).

CHART
NO. 3

PORTABLE HARRIS CELL FOR TRI-AXIAL TESTS

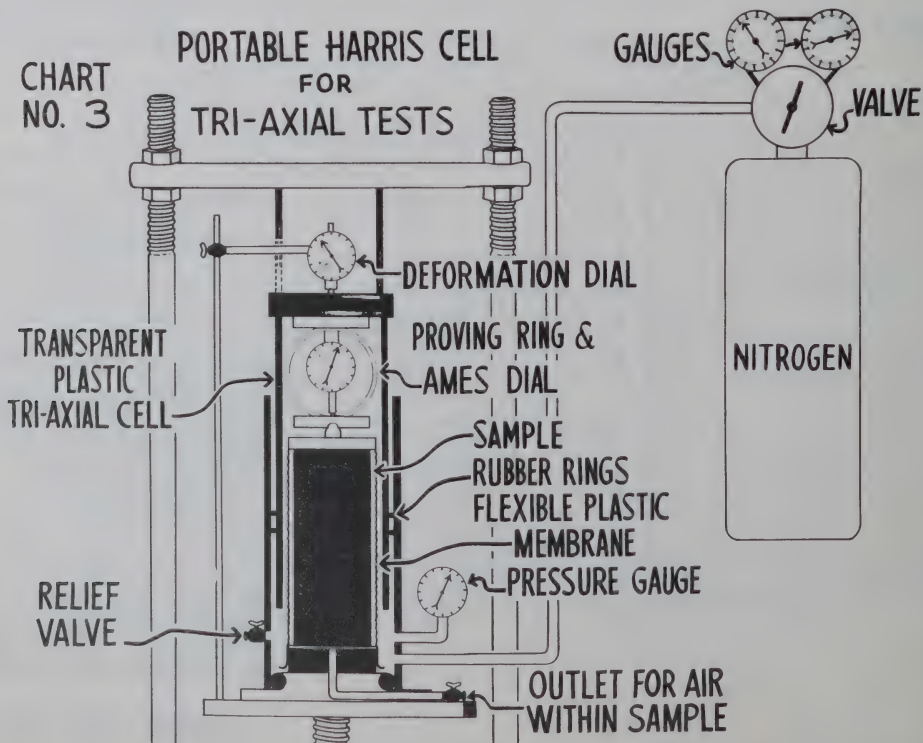


CHART NO. 4

AVAILABLE STRENGTH BASED ON SOIL TEST DATA -LBS./SF.-

~ USING A SAFETY FACTOR OF 2.0 ~

0 200 400 600 800 1000 1200

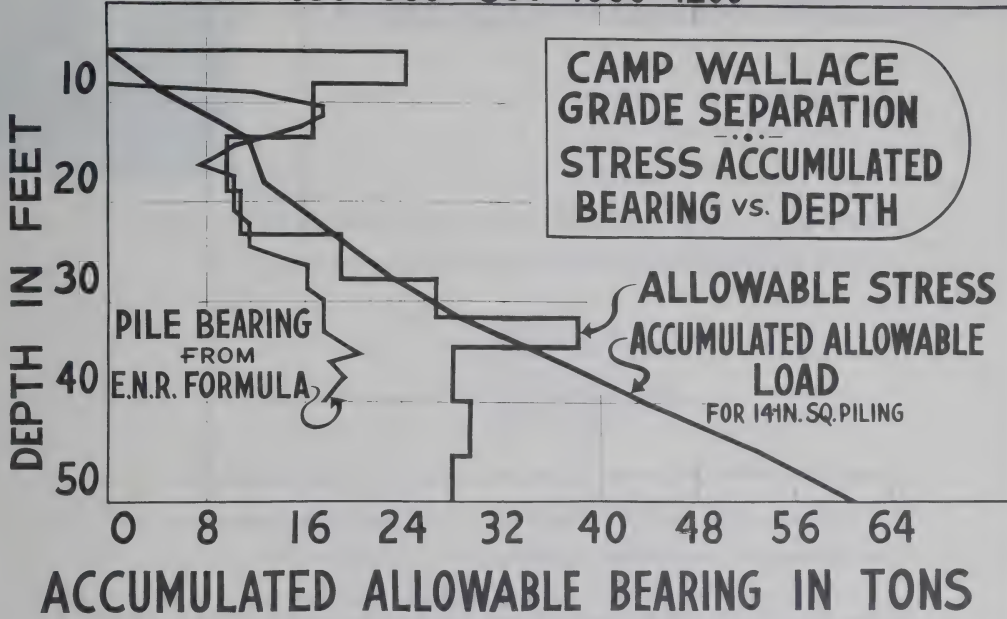
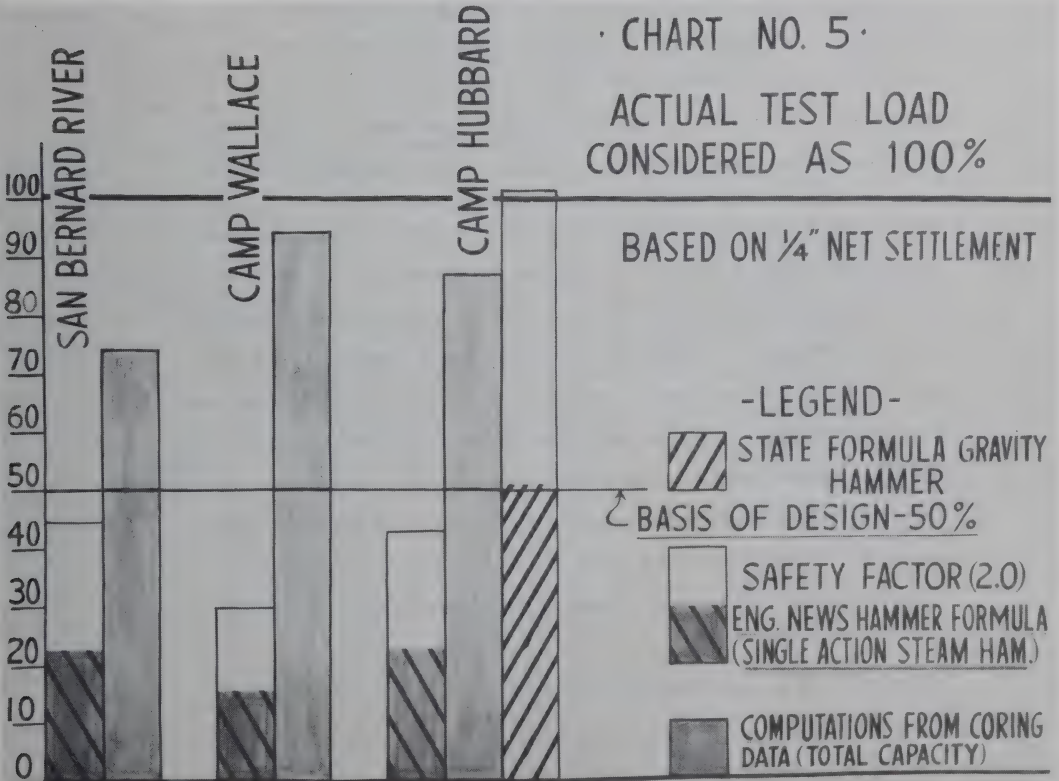


CHART NO. 5

ACTUAL TEST LOAD
CONSIDERED AS 100%

BASED ON 1/4" NET SETTLEMENT



11:20 Discussion
M.D. Shelby, Senior Soils Engineer
Texas Highway Department
Austin, Texas



PROGRESS REPORT ON THE CORRELATION OF THE STATIC LOAD CAPACITY
OF A PILE WITH THE SHEARING STRENGTH OF THE SURROUNDING SOIL. *

by
M. D. Shelby
Senior Soils Engineer
Bridge Division
Texas Highway Department
Austin, Texas

I would not pretend to be able to properly discuss, in ten minutes, the fine paper Mr. Kyser has just presented so I will confine my remarks to a few points that appear to be important.

Several of us in the Bridge Division have long seen the need for a rational method of predetermining the lengths for friction piles as well as a substitute for the old unreliable dynamic hammer formula in determining friction pile capacity. Mr. Kyser's paper brings us up to date on the progress we have made within the Department toward answering these needs. It now appears to those of us who have been closely associated with this development work that the general method he has outlined can and should be used in those instances where friction piles are involved and where satisfactory samples can be obtained and tested.

As a transitional step it would not be our recommendation that the dynamic hammer formula be eliminated from the specifications but should be retained and modified to fit the known soil conditions for each project. This would be done by driving the first pile for a given structure adjacent to a test boring where the strengths of the various soil strata have been predetermined. In other words the soil strength determinations would displace the presently accepted static load test with its many inconvenient delays to the contractor and headaches for the Resident Engineer and his personnel. After driving the first pile and determining a suitable modification for the dynamic formula it would then be used as a guide

*Discussion of a paper of this title by Mr. A. C. Kyser, Assistant District Engineer.

in driving the remainder of the piles, not as a means of measuring pile resistance but rather as a check on possible non-uniformity of the existing earth strata not reflected by the preliminary exploration data. In proposing the adoption of this change in method of determining pile capacity we are not unmindful of the difficulties involved, the limitations of the method and the precautions that will be required for conservative design practices. The following elements are considered essential for the successful use of the method:

1. Adequate foundation exploration which includes undisturbed sampling.
2. Sufficient number of tests to establish the shear strength of each significant soil strata involved.
3. Correct interpretation and application of the test results in the design.
4. An efficiently operated District Laboratory.
5. An alert inspection force during construction.

The adoption of this soil strength method of measuring pile resistance would not preclude the use of the dynamic hammer formula in the many instances where it gives satisfactory results. The decision as to which method is to be used on a given project, however, should be based only upon factual data obtained from adequate foundation explorations. In general, the use of the dynamic hammer formula should be confined to locations where the piling will be practically point bearing or on small structures where exploration data indicates that dynamic resistance will be adequate at reasonable penetrations.

Our experience to date indicates that where the major soil strata thru which the pile is to be driven has a Penetrometer value of less than about 35 blows per foot you may expect that the dynamic resistance will be low and will not build up appreciably with additional penetration. It is in localities where this condition exists that the dynamic hammer formula fails us completely and the soil strength method proves most valuable. The Penetrometer Test gives us a rule of thumb method for

classifying soils and is especially valuable on small Farm Road structures where a complete series of soil strength tests might not be justified if ample dynamic resistance could be obtained at or near the minimum pile penetration required for lateral stability.

The so called Soil Strength method of determining the capacity of a pile is by no means new to the engineering profession. There are several good articles in print on the general theory involved. Outstanding among them and recommended to anyone interested is a paper by Mr. William W. Moore entitled, "Experiences in Predetermining Pile Lengths" and published in the 1949 A.S.C.E. Transactions. Additional valuable correlative data is contained in a report published in 1950 by the Nebraska Department of Roads covering static load tests on steel shell piles for the "Q" Street Viaduct in the City of Omaha.

The development and research work on the overall problem is by no means complete. Where wide differences exist in the Modulus of elasticity of the various supporting earth strata it could logically be argued that the firmer strata might have to supply more than their share of resistance because the deflection of the pile might not be enough to mobilize the full resisting forces in the softer strata. Our present thought is to overcome this difficulty by discounting partly or entirely the load carrying capacity of these soft formations where they occur in connection with firm earth strata. Also ever present when dealing with these soft formations is the possibility of settlement from long time consolidation. This is a problem which is present regardless of the method of measuring pile resistance but is much less hazardous where good exploration data is available for study.

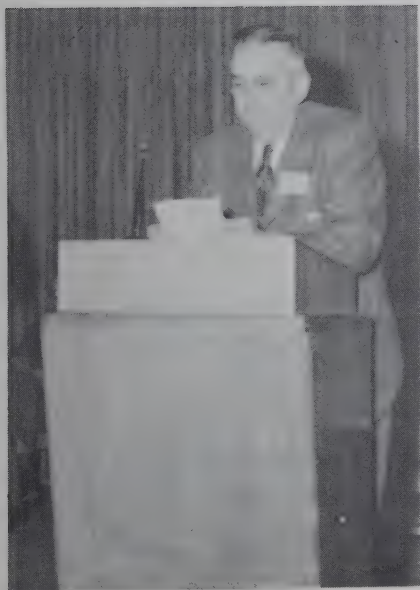
A study of the principles involved in this soil strength method of pile capacity determination suggests the same general approach can logically be applied in calculating the capacity of the Drilled Shaft type of substructure. Some work has been done in this direction by the Houston Urban Expressway organization. However, a more complete study with full

experience and improvement in our technique, especially in the methods of undisturbed sampling, will undoubtedly result in a wider usage of the method by the engineering profession.

Recognition is due Mr. Douglas and his personnel for their fine work in this field. Mr. Kyser and the District 12 Laboratory personnel as well as the District 20 Laboratory at Beaumont have made significant contributions to the cause. Thanks are also due the Soils Section of the Materials and Tests Laboratory at Austin for their many assists and to the excellent spirit of co-operation shown by the Camp Hubbard Shops of the Equipment Division in the development of the soil sampling devices.

Let's not forget there are two kinds of pile foundation failures ---- Physical and Economic. Fortunately we have had very few of the first kind in the Texas Highway Department. I believe we can greatly reduce the chances of having either kind by further development of and a proper application of the principles outlined in Mr. Kyser's paper.

• • •



11:30 General Aspects of the Baytown Tunnel
Jim Douglas, District Engineer
Texas Highway Department
Houston, Texas



GENERAL ASPECTS OF THE BAYTOWN TUNNEL

by

Jim Douglas, District Engineer
Texas Highway Department

For many years there was from time to time a movement on foot by interested parties, including the Navigation District of Harris County, to eliminate three ferries forming the only vehicular crossings of the Houston Ship Channel between the Turning Basin and Galveston Bay. With the vast amount of ship movement on this channel these ferries constitute a very definite hazard. The ferry at Pasadena has now been replaced by the Washburn Tunnel built by Harris County. Construction by the State has been underway for more than two years on the Baytown Tunnel. This subaqueous vehicular tunnel is located approximately equidistant from the Lynchburg and Morgan's Point ferries and will eliminate both of them. This location was determined from traffic and physical studies.

After determining the location hydrographic and soil surveys indicated that the trench-and-precast type of tunnel was most adaptable. This method of construction is performed by casting the tunnel in sections, placing temporary bulkheads in both ends of each section, towing these sections to their respective locations, sinking into a prepared trench dredged to line and grade and joining the sections rigidly by specially designed joints. This method eliminates the use of compressed air and reduces diver work to a minimum.

In the design stage the State's Consulting Engineers, Parsons, Brinckerhoff, Hall and MacDonald considered the conventional octagonal cross section type of tunnel and compared it with the less used circular cross section. These section are shown on the print displayed here. In transverse cross section the Baytown tunnel differs from previous subaqueous vehicular tunnels in that it consists of a circular exterior steel shell, reinforced by internal stiffener rings and ring girders, and a reinforced concrete lining.

Both types were included in the plans and alternate bids were received. The circular section was sixteen percent under the octagonal. The principal difference is in the design of the strengthening and construction problems involved. Apparently the economy offered by the circular section is due to:

- (a) Simplified shell fabrication resulting in low welding costs.
- (b) Elimination of tremie concrete jacket and attending form work.
- (c) Because all concrete lining is placed inside the steel shell in the dry, delays due to weather are eliminated.
- (d) Designing the concrete ring thicker at the invert results in greater stability and added equilibrium of the floating tube during interior concreting operations.
- (e) Economical use of materials and ease of fabrication.

The over-all length of the Baytown tunnel is 4111 feet consisting of an open approach of U-shape cross section 551 feet long at each end and 3009 feet of tunnel proper between the portals. The portion between portals involves two methods of construction. First the trench and precast method comprising nine sections 34 feet 10 inches in diameter, six of which are 300 feet long and three 250 feet. While these sections were being fabricated the trench was dredged to a maximum depth of 91 feet below Mean Low Water into which the tubes later were lowered to line and grade in continuous sequence from north to south and joined together. The second method of construction covers 225 feet at each portal to be constructed in the dry and known as cut and cover method. Near the north portal, the ventilation building is an integral part of the tunnel portal section. The U-shaped open approaches also will be built in the dry.

The work has been let in two contracts. The first contract covered the subaqueous tubes, the dredging of the trench and placing this excavation in the roadway approaches across Black Duck bay, Blackwell peninsular, San Jacinto bay and Spillman island. These fills built to elevation nineteen required the 750,000 cubic yards of trench excavation plus 500,000 cubic yards of hydraulic borrow. Bridges 165 feet long were built in each bay to allow for navigation by small

craft and prevent stagnation of bay water. This contract has been completed at a cost of slightly more than \$5,000,000.00. Contract II was let during the past fall in the amount of approximately \$4,000,000.00. This work includes the cut and cover sections, the open approaches, the tile lining throughout the tunnel proper, the ventilating building, the mechanical and electric equipment, the repair and fire truck garage near the north portal and a crash truck garage near the south portal.

Returning now to the completed contract, I will outline the general sequence of construction operations which I will attempt to show more clearly by slides later. The steel outer shell of the nine subaqueous tubes were fabricated on ways at the shipyards of the Consolidated Western Steel Company at Orange, Texas. The ends of the tubes were closed by steel bulkheads, launched into the Sabine river partially ballasted by reinforcing steel to be used later in the reinforced concrete lining. The tubes were then towed through the intracoastal waterway, Galveston bay and Houston Ship Channel and moored in a cove on Spillman Island. This cove was formed by widening the tunnel trench. The concrete lining was poured by Pump Crete method through hatches in the top of the tube. Usually there were three tubes moored along side the plant dock and work carried on simultaneously. Care had to be taken to make the pours of concrete in such manner as to not overstress any portion of the steel outer ring which was floating at all times. Also care had to be taken to prevent too much rotation of these floating tubes by loading one side too heavily. These operations were carried on until the top of the tube was almost awash. At this point the tubes were towed to a prepared place, low tower manholes fastened to hatches and ballasted to negative buoyancy by placing enough sand and gravel on the roadway slab to complete inside concreting and river sand underneath the roadway in the fresh air duct.

Especially designed barges with steam hoists were moved over the submerged tube. Lines were made fast to hangers on the outside of tube. It was then picked up by the steam hoists and towed to position and let down on a subgrade prepared of three feet of fine gravel screeded to proper line and grade. Usually very heavy precast concrete weights were placed on top of the tube to guarantee that it would not float due to change in specific gravity of the water in the

much polluted channel. Select soil backfill was then carefully placed halfway up on each side of the tube. Then any available soil was placed over the top in such manner as to not flow around the tube ends. This procedure was then repeated with each succeeding tube.

Mention was made above of the changing specific gravity of the ship channel water. This gave the Contractor more concern than any other problem. Each of the tubes when ballasted weighed approximately 9000 tons. On occasion of sinking one of the center tubes, a diver took a sample from near the bottom of the trench at 6:00 A.M. The specific gravity was slightly over one. The tube was ballasted to 350 tons negative buoyancy, which was ample. About 10:30 A.M. the tube was lowered. At five feet above grade it would go no further. Another sample was taken and the specific gravity had increased to 1.06. Six hundredth of 9000 tons is 540 tons, or greater than the negative buoyancy by 190 tons, so naturally the tube became static when negative buoyancy and weight of water displaced were the same. The tube was raised, towed to mooring and the dredge began changing water at the bottom of the trench. A week later the tube was lowered, partially connected to the previously sunken tube and partially backfilled. The next morning the tube was floating again. This caused a delay of a month while backfill was removed, the gravel subgrade repaired and screeded to grade.

The U-shaped approach sections at each end of the tunnel will be supported on Raymond concrete piles. The grade line in these sections drops from elevation plus 19 feet to minus about 20 feet. Although the retaining wall and roadway floor are rather massive, there will be sufficient uplift when storm tides occur to change the piling from foundation to anchor piling. This requires that both load tests and pull tests must be conducted during the driving of these piling.

The portion of the cut and cover section near the north portal, which is integral with the ventilation building, will be supported on timber piling. This building will be of three stories above the tunnel top and will be approximately 50 feet above the surrounding ground. It will house three fresh air fans, each with a rated capacity of 300,000 CFM. Only two of these fans are required for normal operation, the other is for standby in case of failure. Control is automatic, and includes continuous air sampling and recording devices.

Carbon monoxide analyzing equipment provides means for measuring the carbon monoxide content of the tunnel air near each portal. Interconnected with a single point strip chart recording potentiometer, this equipment produces a continuous graphic record of the concentration of carbon monoxide at each sampling point. Operation of the ventilation system is such that the concentration of carbon monoxide is kept below 4 parts in 10,000 - a proportion which is exceeded at times on busy street intersections. The fans feed fresh air through a single air duct underneath the roadway and from it through fresh air ports spaced longitudinally at 16-foot intervals on either side just above the roadway. The fresh air dilutes the vitiated air and exhausts it through the portals.

Power is furnished for tunnel operation by two 12,000-volt incoming feeders terminating at substations located in the ventilating building. An emergency motor generator set is also provided in event of power failure. Tunnel lighting is accomplished by a continuous slimline fluorescent system throughout the length of the tunnel. These lights consist of two rows of fixtures mounted on the ceiling profile over the center of the roadway. To provide transition from bright sunlight to tunnel lighting, two additional rows of lights are mounted parallel to and on each side of the main tunnel lights for some distance inward from the portals. In case of power failure, provision is made so that a portion of the lighting system becomes automatically connected to the emergency motor generator source of power.

At intervals of 375 feet on the sidewalk side are located heavy duty push-button switching equipment so that patrolmen can control red-amber-green traffic signals for each lane of traffic for proper movement of vehicles. Should it become necessary to stop incoming traffic at some point in the tunnel, the patrolmen by pushing the proper button at the nearest control station can change green lights to red facing incoming traffic but leave all signals beyond the trouble spot green for traffic already past the point to proceed out of the tunnel.

A sound-powered telephone system is provided as an integral part of the tunnel operational procedure. This installation enables complete intercommunication at all times between the control room, the tunnel and garages. Radio reception and transmission will be provided so there will be no cut-out of program from any

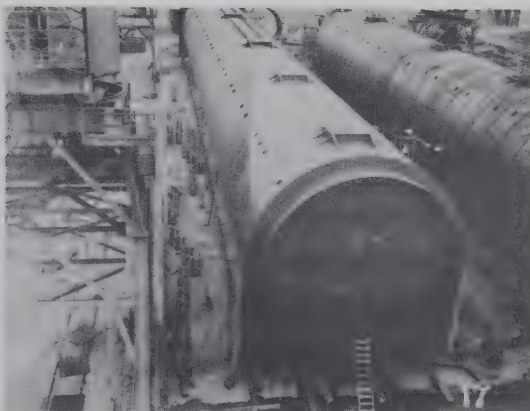
of the local radio stations while a motorist is passing through the tunnel.

I have tried to touch a little on design and a little on construction.

Either properly covered would require a very long paper. Now we will see a few slides made during the construction of Contract I.



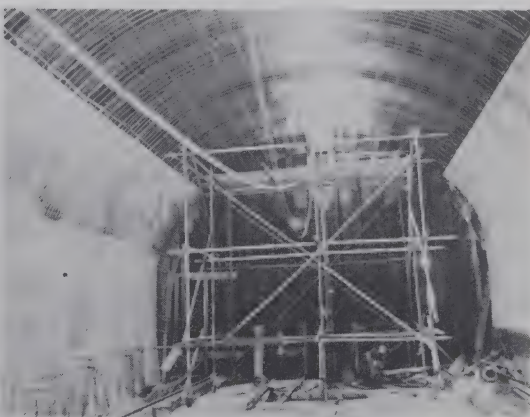
Tube on ways at Consolidated Steel Corporation, Orange, Texas.
End launching position.



Foreground view - River end of ways
northerly end of tube with bulkhead.
Dam plate and collar complete for joint
to open cut and cover portion of tunnel.



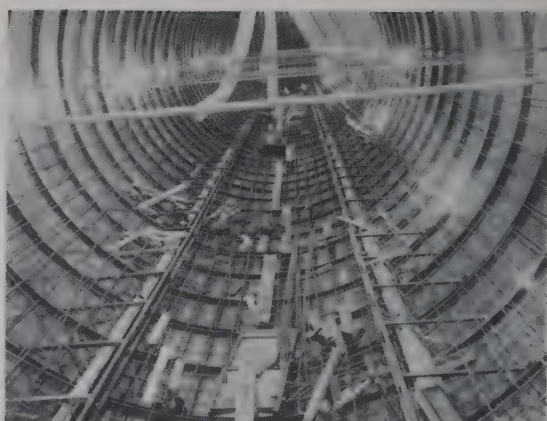
Inside tube - First pour of invert and
sidewalls in place.



Tube south - Concrete tunnel lining
complete through pour 8. Reinforcing
steel in place for pour 9.



Tube, view north - Tunnel lining complete through pour 8.



Section I reinforcing steel and forms for number 1 pour in place.



Tube in ballasting area - Alignment towers in place, access towers being set prior to ballasting.



View looking north showing part of roadway beams in place. Pour 5 and electrical conduit on west side in place.



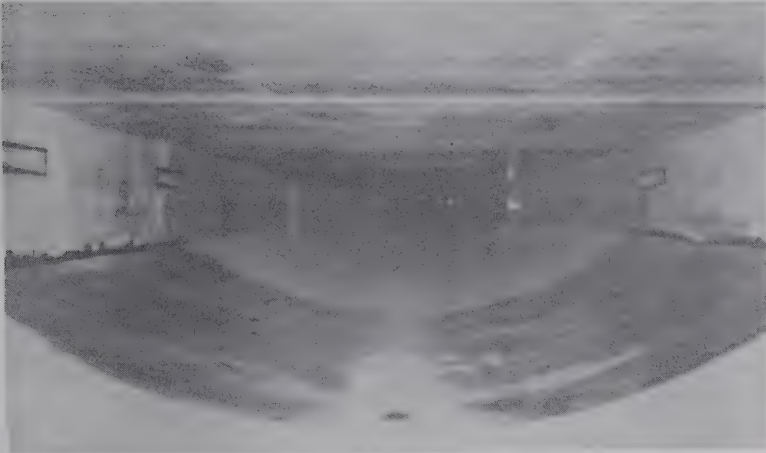
Foundation course screed scow, with carriage and track set up to screed on 1 per cent grade.



Tubes looking southward from dredge temple to berthing area.



Bulkhead partially removed at joint.



View from north end of tube in air duct looking north, showing joint and pump room deflection walls.



View from tube looking north.

12:00 Noon - Adjournment





Highway employees attending the General Staff Meeting on March 4, prior to the Short Course.

SHORT COURSE ROSTER

ACKER, Loyd E., Austin
Engineering Assistant (THD)
ADKINS, Marvin C., Dallas
Sr. Design. Engineer (THD)
ALEXANDER, Randle B., Austin
Bridge Engineer (THD)
ARNO, Howard L., Austin
Director of Personnel (THD)
ARNOLD, D. C., Houston
The Texas Co.
ARNOLD, Ernest M., Fort Worth
Bureau of Public Roads
ASHBY, J. C., Denton
Maintenance Foreman (THD)
ASKEW, David H., Austin
Sr. Design. Engineer (THD)
ATKINSON, Claude, Pleasanton
Sr. Maintenance Foreman (THD)

BAER, John A., Seguin
Senior Inspector (THD)
BAILEY, Percy S., Austin
Maintenance Engineer (THD)
BAKER, James L., Angleton
Sr. Resident Engineer (THD)
BAKER, Terry A., Pharr
Sr. Design. Engineer (THD)
BALLARD, Richard P., Abilene
Portland Cement Assn.
BARTEL, Herbert H., Dallas
S.M.U. Instructor
BATSON, Lee R., Rosenberg
Chief Inspector (THD)
BATTLE, Joe M., Houston
Sr. Design. Engineer (THD)
BEAUCHAMP, Brawley O., Pecos
Sr. Party Chief (THD)
BECKHAM, Joe W., Lufkin
Texas Foundries, Inc.
BELL, Tyree, Dallas
Austin Road Co.
BENNETT, Fred T., Austin
Engineer, Traffic Services (THD)
BENSON, Fred J., College Station
A. & M. College
BERTRAM, Jack L., Fort Worth
Resident Inspector (THD)
BEST, George F., Jasper
Resident Designer (THD)
BLACK, G. W., Stephenville
Resident Designer (THD)
BLACKALLER, J. H., San Antonio
H. B. Zachry Co.
BLACKSTONE, Robert B., Austin
Jr. Draftsman (THD)
BLAIR, John E., Bryan
District Engineer (THD)
BLASINGAME, John C., Garland
Sr. Landscape Adviser (THD)
BLUESTEIN, Ed., Atlanta
District Engineer (THD)

BOSSY, R. A., San Antonio
Expressway Engineer (THD)
BOSTICK, Jack P., Austin
Section Supervisor (THD)
BOYD, A. W., Fort Worth
Dist. Maintenance Engineer (THD)
BRADFORD, R. M., Longview
Sr. Resident Engineer (THD)
BRADLEY, Rhea E., Littlefield
Engineering Assistant (THD)
BRANNAN, J. B., Breckenridge
Resident Engineer (THD)
BRAUNIG, Carl F., San Antonio
Engineering Assistant (THD)
BROWN, C. H., Orange
Sr. Resident Engineer (THD)
BROWN, Joe T., Belton
Sr. Resident Engineer (THD)
BROWN, Martin O., Houston
Engineering Assistant (THD)
BROWN, Norman J., San Antonio
Resident Inspector (THD)
BROWN, Robert A., Weatherford
Engineering Assistant (THD)
BRYAN, John H., Jacksboro
Engineering Assistant (THD)
BURKHEAD, Hubert, Fort Worth
Junior Inspector (THD)
BURNS, Jr., Robert E., Marshall
Resident Engineer (THD)
BURTNER, B. P., Paris
Sr. Resident Engineer (THD)
BUSBY, S. R., Pharr
Assistant District Engineer (THD)
BUSHFIELD, W. C., San Antonio
Sr. Resident Engineer (THD)

CABANISS, L. D., Paris
District Engineer (THD)
CAGLE, C. C., Sherman
Sr. Resident Engineer (THD)
CALFEE, Benjamin C., Conroe
Senior Inspector (THD)
CALVIN, E. B., Dallas
District Construction Engineer (THD)
CAMPBELL, Charles, Beaumont
Resident Inspector (THD)
CAMPBELL, Edwin M., Rusk
Resident Engineer (THD)
CAMPBELL, M. Earl, Arlington
Highway Research Board
CARDWELL, Richard N., Austin
Sr. Laboratory Assistant (THD)
CARL, Ed M., Austin
Sr. Design. Engineer (THD)
CARLTON, Don N., San Angelo
Office Engineer (THD)
CARMICHAEL, W. E., Houston
District Construction Engineer (THD)
CARROLL, Joe E., San Antonio
Uvalde Rock Asphalt Co.

CARTER, H. C., Austin
District Construction Engineer (THD)
CARTER, LeRoy C., Mineola
Resident Engineer (THD)
CARVER, George L., Austin
Inventory Manager (THD)
CARY, J. F., Austin
Bureau of Public Roads
CAWTHON, Frank W., Dallas
District Engineer (THD)
CHAFFIN, C. Weldon, Austin
Sr. Chemical Engineer (THD)
CHAPMAN, James T., Karnes City
Resident Engineer (THD)
CHOLLAR, Allen L., Austin
Supv. Design. Engineer (THD)
CHRISTIAN, Archie H., Austin
Sr. Design. Engineer (THD)
CLARK, Joe C., Bonham
Sr. Resident Engineer (THD)
CLARK, Maurice H., Austin
Associate Resident Engineer (THD)
CLARK, Pat W., Austin
Supv. Design. Engineer (THD)
CLARK, William A., Richmond
Senior Draftsman (THD)
CLIFTON, Burton, Mineral Wells
Engineering Assistant (THD)
CLOPTON, Albert H., Austin
Area Supervisor (THD)
COBURN, Maurice, Plainview
Associate Resident Engineer (THD)
COKER, Garland, Austin
Area Supervisor (THD)
COLE, Kenneth H., McKinney
Sr. Party Chief (THD)
COLEMAN, Paul H., Pecos
Assistant District Engineer (THD)
COLLIER, Thomas C., Austin
Asst. State Highway Engineer (THD)
COLLINS, Edward S., Dallas
Trinity Portland Cement Co.
COLLINSWORTH, K. D., Austin
Landscape Adviser Assistant (THD)
COOPER, B. W., Conroe
Resident Engineer (THD)
CORNELIUS, M. G., Yoakum
District Engineer (THD)
CORREA, Jose, Houston
Associate Design. Engineer (THD)
CORTEZ, A. J., Port Arthur
Resident Inspector (THD)
COULTER, B. W., Austin
Hot Mix Association
COURTER, John W., Fort Worth
Bureau of Public Roads
COX, Clifford E., Austin
Associate Office Engineer (THD)
COX, Samuel G., Pharr
Engineering Assistant (THD)

COX, Sidney C., Mineola
Engineering Assistant (THD)
CRAIN, Oscar L., Bryan
Sr. Resident Engineer (THD)
CRAVENS, James W. Paris
District Maintenance Engineer (THD)
CRAWFORD, Jim, Mt. Pleasant
Sr. Resident Engineer (THD)
CROWE, James R., San Augustine
Resident Engineer (THD)
CRUTCHER, I. H., Tyler
District Maintenance Engineer (THD)

DABNEY, Gordon W., San Antonio
Engineer Sect. Hdq. 4th Army
DAVENPORT, John M., Austin
Administrative Aide (THD)
DAVIS, F. M., San Antonio
District Engineer (THD)
DAVIS, Thomas C., Corpus Christi
District Engineer (THD)
DAVIS, W. A., Sanderson
Sr. Resident Engineer (THD)
DAY, Glenn M., Mansfield
Resident Inspector (THD)
DE BERRY, B. L., Greenville
Sr. Resident Engineer (THD)
DE BERRY, Henry D., Austin
Administrative Analyst (THD)
DEEN, Harvey E., Austin
Engineer of Aid Projects (THD)
DEGAN, Carl, Orange
Resident Inspector (THD)
DELK, F. M., Houston
Supv. Construction Engineer (THD)
DE MAURI, J. A., Kingsville
Chief Inspector (THD)
DE SHAZO, John J., Dumas
Engineering Assistant (THD)
DEWALD, J. C., Amarillo
Resident Engineer (THD)
DEWEY, Cooper, Houston
Sr. Resident Engineer (THD)
DIAL, Raymond W., Fort Worth
Worth Construction Co.
DILLON, Ben E., Brady
Resident Engineer (THD)
DINGWALL, J. C., Austin
Engineer, Road Design (THD)
DOCKERY, W. D., Austin
District Engineer (THD)
DODSON, John R., Jasper
Engineering Assistant (THD)
DOKOS, Nick George, Houston
Engineering Assistant (THD)
DOOLEY, A. J., Corsicana
Sr. Maintenance Foreman (THD)
DOSAMENTES, Armando
Irrigation Bur., Rep. of Mexico
DOSHIER, Douglas R., Childress
Sr. Maintenance Supt. (THD)
DOUGLAS, Jim, Houston
District Engineer (THD)
DOUGLAS, Jap H., McCamey
Maintenance Foreman (THD)
DRAKE, Glynn W., Austin
Chief of Field Audits (THD)
DRAKE, James E., Austin
Procurement Expediter (THD)

DREWS, Leonard W., Austin
Area Supervisor (THD)
DUNWOODY, Cullen B., Tyler
Sr. Party Chief (THD)

EATMAN, Arthur W., Austin
Asst. Materials & Tests Engr. (THD)
EDWARDS, C. C., Beaumont
Associate Office Engineer (THD)
EDWARDS, G. G., Austin
Uvalde Rock Asphalt Co.
EDWARDS, Ted L., Austin
Portland Cement Association
EFFENBERGER, Ewald J., Edna
Resident Engineer (THD)
EHLERS, Walter E., San Antonio
Resident Engineer (THD)
EHLINGER, Leo, Brownwood
Retired District Engineer (THD)
ELDRIDGE, Herbert, Austin
Chief Engineer of Planning (THD)
ESTES, Elray W., Liberty
Sr. Office Assistant (THD)
EVANS, Elton B., Groesbeck
Resident Engineer (THD)

FAGALA, Duwain E., San Juan
Engineering Assistant (THD)
FALTONSON, Ray C., Austin
Engineer-Director, Operations (THD)
FALTONSON, R. L., Pecos
District Engineer (THD)
FAUST, William R., San Antonio
Sr. Design. Engineer (THD)
FERRIS, Jr., G. Thomas, Houston
Chief Inspector (THD)
FINCHER, R. H., Austin
Supv. Urban Engineer (THD)
FINLEY, W. W., Fort Worth
District Engineer (THD)
FISHER, Thomas A., Rosenberg
Associate Resident Engineer (THD)
FOSTER, Thomas O., Corpus Christi
Asst. District Engineer (THD)
FRANK, Bernie L., Atlanta
Sr. Resident Engineer (THD)
FRANK, Jr., Charles H., Pharr
Senior Draftsman
FRANKE, E. G., San Antonio
Uvalde Rock Asphalt Co.
FRAZER, Donald V., Henderson
Senior Draftsman (THD)
FRAZIER, John H., Austin
Bureau of Public Roads
FREEBOROUGH, B. B., Austin
Engineer, Land Service Roads (THD)
FRY, Glen, Sulphur Springs
Sr. Resident Engineer (THD)
FURRY, Harlan, Beaumont
Designing Engineer (THD)

GALLAWAY, Frank D., Livingston
Resident Engineer (THD)
GALLEGLY, Henry L., Dalhart
Sr. Party Chief (THD)
GARCIA, Geronimo G., Laredo
Engineering Assistant (THD)
GARRARD, William L., Austin
Office Engineer (THD)

GARREN, M. C., Austin
Sr. Information Clerk (THD)
GARRETT, George M., Fort Worth
Engineer-Manager (THD)
GEMMER, E. Phil, Houston
Texas Construction Materials Co.
GENTRY, A. C., Tyler
District Engineer (THD)
GILCHRIST, Gibb, College Station
Chancellor, A & M College System
GILLETTE, H. S., Fort Worth
Bureau of Public Roads
GOODE, Mark G., Dallas
Resident Engineer (THD)
GOODE, Preston C., Austin
Sr. Administrative Engineer (THD)
GOODMAN, Walt P., Dallas
Lone Star Cement Corp.
GOODRUM, Forest W., Houston
Engineering Assistant (THD)
GONZALEZ, Jose L., Laredo
Engineering Assistant (THD)
GORDON, Arthur M., Fort Worth
Supv. Design. Engineer (THD)
GRAHAM, Jno. M., Navasota
Sr. Resident Engineer (THD)
GREGORY, Herbert, San Antonio
Resident Inspector (THD)
GREEN, Grover, Dallas
Dist. Maintenance Engineer (THD)
GREEN, John L., Austin
Director, Claims (THD)
GREER, D. C., Austin
State Highway Engineer (THD)
GREER, M. V., Austin
Supv. Traffic Engineer (THD)
GRIFFITH, Paul Bing, Austin
Designing Engineer (THD)
GRISSOM, Harold H., Cleburne
Assoc. Resident Engineer (THD)
GROVES, Algie, Matador
Resident Inspector (THD)
GRUBB, Clayton, Angleton
Resident Inspector (THD)
GUINN, R. S., Cedar Valley
Sr. Field Engineer (THD)
GULLION, Guy, Atlanta
Resident Engineer (THD)
GUSTAFSON, Wilfred F., Austin
Sr. Design. Engineer (THD)

HALIBURTON, Tracey, Pharr
Sr. Maintenance Supt. (THD)
HALL, Grainger L., Greenville
Engineering Assistant (THD)
HAMM, J. G., Fort Worth
Assoc. Office Engineer (THD)
HAMMOND, Wilton N., Fort Worth
Engineering Assistant (THD)
HAMNER, Bob, Austin
Administrative Analyst (THD)
HANK, R. J., Austin
Chief Engineer of Operations (THD)
HANOVER, Joe G., Hearne
Resident Engineer (THD)
HARDWICK, Jack C., George West
Chief Inspector (THD)
HARKLEROAD, Henry, Austin
Asst. Bridge Engineer (THD)

HARKRIDER, D. T., Austin
 Director, Insurance (THD)
 HARLAN, Herman V., Crockett
 Sr. Resident Engineer (THD)
 HARPER, W. A., Kaufman
 Sr. Resident Engineer (THD)
 HARRIS, John R., Austin
 Supv. Construction Engineer (THD)
 HARRIS, F. A., Houston
 Sr. Design. Engineer (THD)
 HARVEY, J. Nolan, Austin
 Southwestern Materials
 HASTINGS, Clinton B., Austin
 Sr. Laboratory Assistant (THD)
 HAYES, Milton, Beaumont
 Sr. Maintenance Supt. (THD)
 HECK, George, San Antonio
 4th Army H.Q., U. S. Army
 HEIM, John L., Austin
 Asst. Dir. Equip. & Procurement (THD)
 HELEMAN, Herbert W., Austin
 Area Supervisor (THD)
 HENDRICKSON, O. W., Dallas
 Austin Road Co.
 HEUSER, Julius F., Houston
 Resident Inspector (THD)
 HILGERS, Herbert F., San Antonio
 Sr. Resident Engineer (THD)
 HILL, Arthur J., Austin
 Asphalt Engineer (THD)
 HILL, John W., College Station
 Texas A & M College
 HILL, Walter N., Dallas
 Austin Paving Co.
 HIX, Charles, Jr., Tyler
 Senior Draftsman (THD)
 HODGES, M. B., Del Rio
 District Engineer (THD)
 HODGSON, Alonzo H., Jr., Atlanta
 Chief Inspector (THD)
 HOGAN, David L., Bryan
 Dist. Maintenance Engineer (THD)
 HOGAN, David W., Tyler
 Senior Draftsman (THD)
 HOLMES, Charles T., San Antonio
 Asst. District Engineer (THD)
 HOLMES, Oscar W., Austin
 Resident Engineer (THD)
 HOLT, Fred D., Dallas
 Designing Engineer (THD)
 HOOPER, Dwight W., Seguin
 Sr. Resident Engineer (THD)
 HORNSBY, Hugh M., Austin
 Field Engineer (THD)
 HOUSTON, George A., Brownsville
 Sr. Resident Engineer (THD)
 HUDSON, H. S., Austin
 Servtex Materials Co.
 HUDSON, Raymond P., Gilmer
 Senior Instrumentman (THD)
 HUFF, Sam L., Austin
 Sr. Field Engineer (THD)
 HUFF, Talbot S., Austin
 Assistant Engineer, Road Design (THD)
 HUFFMAN, Tom, Dallas
 Dallas Chamber of Commerce
 HUGHES, Chuck H., Austin
 Apprentice Engineer (THD)

HUNTER, Thomas G., Carthage
 Resident Engineer (THD)
 HUTCHISON, A. D., Paris
 Assist. District Engineer (THD)
 HVEZDOS, Steve S., Longview
 Resident Engineer (THD)

INGRAM, Lon C., Corsicana
 Sr. Resident Engineer (THD)
 INGRAM, Temple B., Gilmer
 Sr. Resident Engineer (THD)
 JACKSON, J. F., College Station
 A & M Methodist Church
 JANAK, Clarence J., Temple
 Senior Inspector (THD)
 JARL, Duval C., Austin
 Administrative Aide (THD)
 JEFFERSON, G. T., Fort Worth
 Assoc. Resident Engineer (THD)
 JELINEK, E. A., Austin
 Sr. Design. Engineer (THD)
 JENNINGS, R. N., San Angelo
 Dist. Maintenance Engineer (THD)
 JOHNSON, Allen E., Beaumont
 Sr. Resident Engineer (THD)
 JOHNSON, James R., Abilene
 Designing Engineer (THD)
 JOHNSTON, J. T., Lubbock
 Engineering Assistant (THD)
 JONES, D. T., Lubbock
 Senior Draftsman (THD)
 JONES, John, Seguin
 Sr. Maintenance Foreman (THD)
 JORDAN, M. D. L., San Antonio
 Sr. Maintenance Foreman (THD)
 JUNGMAN, Arthur H., Houston
 Assn. of Am. R.R.S.

KEARBY, Jerome P., Decatur
 Sr. Resident Engineer (THD)
 KEETER, H. C., San Antonio
 Engineering Assistant (THD)
 KELLY, Gene, Pecos
 Resident Engineer (THD)
 KELLY, Harrell J., Beaumont
 Assoc. Resident Engineer (THD)
 KELLY, Jim, Fort Worth
 Engineering Assistant (THD)
 KELLY, Tom J., San Angelo
 District Engineer (THD)
 KESSLER, Herbert W., Austin
 Sr. Office Engineer (THD)
 KIESCHNICK, H. P., Port Arthur
 Resident Inspector (THD)
 KILLMER, R. E., Dallas
 Engineer-Manager (THD)
 KIRBY, Jack H., Brady
 Resident Engineer (THD)
 KIRKPATRICK, J. R., Denton
 Sr. Resident Engineer (THD)
 KOCH, V. G., Lufkin
 District Engineer (THD)
 KOENNING, Jr., A., San Antonio
 Jr. Landscape Inspector (THD)
 KNAPP, George A., Houston
 Assn. of Am. R.R.S.
 KNETSCH, Fred, Seguin
 Former Member, St. Hwy. Comm.

KYSER, Albert C., Houston
 Asst. District Engineer (THD)
 LAAS, Perry J., Yoakum
 Engineering Assistant
 LACKEY, Gerald G., San Angelo
 Sr. Party Chief (THD)
 LANGFORD, Keith, Houston
 Texas Const. Matl. Co.
 LAUGHTER, Jr., John G., Edna
 Senior Draftsman (THD)
 LEDBETTER, Joe O., Weatherford
 Engineering Assistant (THD)
 LEDNICKY, Ben J., Brownwood
 Sr. Landscape Adviser (THD)
 LEO, Walter H., Dallas
 Supv., Steel Ind. Scrap Mobil. Com.
 LEWALLING, W. J., New Braunfels
 Uvalde Rock Asphalt Co.
 LEWIS, Bill, Fort Worth
 Worth Construction Co.
 LEWIS, Robert L., Austin
 Senior Draftsman (THD)
 LIPSCOMB, Lindsey I., Tyler
 Humble Oil & Refining Co.
 LIVINGSTON, J. W., Atlanta
 Chief Inspector (THD)
 LONG, Travis A., Austin
 Resident Engineer (THD)
 LONGORIA, A., Rio Grande City
 Engineering Assistant (THD)
 LOTT, James G., Amarillo
 District Engineer (THD)
 LOVE, A. C., Beaumont
 Ret. Sr. Research Engineer (THD)
 LOVETT, Frank E., Fort Worth
 Assist. Engineer-Manager (THD)
 LOVIL, W. C., Marshall
 Sr. Resident Engineer (THD)
 LUFT, Moritz D., Ingleside
 Chief Inspector (THD)
 LYTTON, R. O., San Antonio
 Supv. Construction Engineer (THD)

MADDOX, Edward F., Dallas
 Sr. Resident Engineer (THD)
 MADDOX, Frank O., Austin
 Lone Star Cement Co.
 MADDOX, Frank S., Austin
 Asst. State Highway Engineer (THD)
 MAGEE, Lawrence, Lubbock
 Office Engineer (THD)
 MALONE, Morris L., Marlin
 Sr. Resident Engineer (THD)
 MANN, G. B., Dallas
 Resident Engineer (THD)
 MARS, E. W., El Paso
 District Engineer (THD)
 MARPLE, L. J., Texarkana
 Resident Engineer (THD)
 MARVEL, Dale D., Houston
 Assoc. Dist. Traffic Engineer (THD)
 MASON, F. B., Corpus Christi
 Sr. Resident Engineer (THD)
 MASSEY, Palmer, Childress
 District Engineer (THD)
 MC CALLA, Kenneth, Austin
 Texas Railroad Assn.
 MC CARTY, S. C., Lubbock
 District Engineer (THD)

MC CRARY, Louis C., Houston
 Designing Engineer (THD)
 MC CULLEY, R. C., Anahuac
 Sr. Office Assistant (THD)
 MC CULLOCH, R.A. Pecos
 District Maintenance Engineer (THD)
 MC DOWELL, Chester, Austin
 Sr. Soils Engineer (THD)
 MC DOWELL, R. F., Austin
 Portland Cement Assn.
 MC GEE, J. C., Cleburne
 Sr. Laboratory Assistant (THD)
 MC KENZIE, J. O., San Benito
 Chief Inspector (THD)
 MC LAIN, Joseph M., Bryan
 Assoc. Resident Engineer (THD)
 MC NAIR, J. T., Fort Worth
 Supv. Construction Engineer (THD)
 MERCER, James R., Decatur
 Engineering Assistant (THD)
 MEYER, George S., San Antonio
 Sr. Resident Engineer (THD)
 MICA, Joseph R., Columbus
 Engineering Assistant (THD)
 MIKSOVSKY, F. G., Angleton
 Resident Engineer (THD)
 MILLER, Viggo, Austin
 Designing Engineer (THD)
 MILLIGAN, Robt. J., Brownwood
 District Engineer (THD)
 MITCHELL, A.F., Corsicana
 Former Member, St. Hwy. Comm.
 MITCHELL, F. S., Houston
 Laboratory Engineer (THD)
 MITCHELL, T. C., Dalhart
 Sr. Resident Engineer (THD)
 MOORE, Howard D., Carthage
 Sr. Instrumentman (THD)
 MONCURE (Miss) Leah, Austin
 Designing Engineer (THD)
 MOEHLMAN, W. F., Gonzales
 Sr. Party Chief (THD)
 MONROE, Morris E., Hamlin
 Assoc. Resident Engineer (THD)
 MORELAND, W. E., Mt. Pleasant
 Chief Inspector (THD)
 MORGAN, J. Shelby, Austin
 Procurement Supervisor (THD)
 MORRIS, L. E., Wichita Falls
 Asst. District Engineer (THD)
 MORRIS, T. B., Pasadena
 Senior Draftsman (THD)
 MOURSUND, A.F., San Angelo
 Ret. District Engineer (THD)
 MULLINS, B.F.K., College Station
 A & M College
 MURPHY, Wallis J., Lytle
 Sr. Party Chief (THD)
 NABERS, John B., Wichita Falls
 District Engineer (THD)
 NABERS, Robt. L., Childress
 Asst. District Engineer (THD)
 NATIONS, John B., Austin
 Dir. Equip. & Procurement (THD)
 NEELLEY, A. G., Austin
 Armco
 NEER, D. M., Austin
 Ret. Supv. Right of Way Engr. (THD)
 NETTLETON, D. A., Dallas
 Supv. Urban Engineer (THD)
 NEWELL, Vance, Austin
 Assoc. General Contractors
 NITTEBERG, C. T., Fort Worth
 Bureau of Public Roads
 ODELL, Earl G., Austin
 Designing Engineer (THD)
 OLSON, Delbert L., Kaufman
 Resident Engineer (THD)
 OPIELA, J. B., Columbus
 Engineering Assistant (THD)
 ORENBAUN, G. T., Big Spring
 Sr. Resident Engineer (THD)
 OWENS, J. M., Austin
 Sr. Resident Engineer (THD)
 PAGE, John M., Austin
 Bureau of Public Roads
 PARMER, E. D., Beaumont
 Assist. District Engineer (THD)
 PARSONS, C. N., Del Rio
 Assist. District Engineer (THD)
 PEEPLES, Bert N., Pecos
 Assoc. Office Engineer (THD)
 PEISKER, F. J., Austin
 Area Supervisor (THD)
 PENDERGRASS, George, Austin
 Director, Accounting (THD)
 PENNYBACKER, P.V., Austin
 Supv. Field Engineer (THD)
 PHILLIPPI, Olof A., Austin
 Assist. Construction Engineer (THD)
 PIPER, James D., Austin
 Portland Cement Assn.
 PITTMAN, E. E., Austin
 Sr. Design. Engineer (THD)
 PITTMAN, J. E., Lubbock
 Assoc. Landscape Adviser (THD)
 POARCH, J. C., Midland
 Resident Inspector (THD)
 POLLARD, Albert H., Austin
 Supv. Field Engineer (THD)
 POORMAN, Omer F., Angleton
 Senior Draftsman (THD)
 POST, E. G., Fort Worth
 Assoc. Resident Engineer (THD)
 POTTER, W. W., Fort Worth
 Assist. District Engineer (THD)
 POTTS, Robt. J., Harlingen
 Member, State Hwy. Commission
 POWELL, Thomas O., Austin
 Bureau of Public Roads
 POWER, J. E., Austin
 Designing Engineer (THD)
 PRICE, Oscar D., Austin
 Bureau of Public Roads
 PRITCHARD, E.M., Brownwood
 Assist. District Engineer (THD)
 PUCKETT, David M., Waco
 District Engineer (THD)
 PUCKETT, John W., Pharr
 Ret. District Engineer (THD)
 PRUETT, Lester J., Lufkin
 Sr. Landscape Adviser (THD)
 RAABE, Martin L., Vernon
 Sr. Resident Engineer (THD)
 RABY, W.C., San Antonio
 Sr. Resident Engineer (THD)
 RALPH, Tom B., Center
 Sr. Resident Engineer (THD)
 RAY, Homer, Snyder
 Resident Engineer (THD)
 RAMERT, Carl V., Yoakum
 Assoc. Resident Engineer (THD)
 REA, Clarence R., Colorado City
 Resident Engineer (THD)
 REAGAN, J. D., Beaumont
 Sr. Design. Engineer (THD)
 REAVES, Fred T., Houston
 Sr. Design. Engineer (THD)
 REASONOVER, C. H., Bonham
 Engineering Assistant (THD)
 REED, Bruce, Fort Worth
 Chief Inspector (THD)
 REGLIN, F., Waxahachie
 Sr. Resident Engineer (THD)
 RENSCHAW, Ralph R., Palestine
 Sr. Resident Engineer (THD)
 RIEFKOGEL, Rudolph, San Antonio
 Sr. Landscape Adviser (THD)
 RICE, J. D., Austin
 Sr. Design. Engineer (THD)
 RICH, Ralph H., Austin
 Hwy. Engr. Bureau of Public Roads
 RICHEY, Jack, Hamlin
 Resident Engineer (THD)
 RIDEOUT, T. R., Jefferson
 Assoc. Resident Engineer (THD)
 RIGSBEE, Hub K., Austin
 Designing Engineer (THD)
 RIHA, Edward A., El Campo
 Engineering Assistant (THD)
 ROANE, Robert T., Katy
 Resident Engineer (THD)
 ROBERTS, J. C., Abilene
 District Engineer (THD)
 ROBERSON, H. G., Dilley
 Sr. Maintenance Foreman (THD)
 ROBINSON, Jed N., Austin
 Construction Engineer (THD)
 ROBINSON, Otis L., Cleburne
 Engineering Assistant (THD)
 ROBY, Lavelle, Midland
 Sr. Party Chief (THD)
 RODMAN, Roy S., Austin
 Supv., Landscape Architect (THD)
 ROTHE, R. L., Lufkin
 Sr. Design. Engineer (THD)
 ROYDER, Jeff P., Houston
 Humble Oil & Refining Co.
 RUCKMAN, C. W., Austin
 Bureau of Public Roads
 RUSSELL, Wylie H., Liberty
 Resident Inspector (THD)
 RUTH, E. M., Corpus Christi
 Sr. Resident Engineer (THD)
 SAFLEY, D. S., Lufkin
 Sr. Resident Engineer (THD)
 SCHNESMAN, Jr., H., Sinton
 Resident Engineer (THD)
 SCHWEERS, B. E., Wharton
 Sr. Party Chief (THD)
 SCRIVNER, Frank H., Austin
 Sr. Research Engineer (THD)

SEARCY, Victor, El Paso
Dist. Maintenance Engineer (THD)
SEEWALD, Emil E., San Antonio
Engineering Assistant (THD)
SEWARD, O. A., Groesbeck
Seward & Shurtleff
SHELBY, J. A., Austin
Sr. Field Engineer (THD)
SHELBY, M. D., Austin
Sr. Soils Engineer (THD)
SHIELS, Tom D., Dallas
Portland Cement Assn.
SHOEMAKER, S. A., Waxahachie
Resident Engineer (THD)
SHORT, Harry H., Cooper
Resident Engineer (THD)
SIDES, Ben C., Andrews
Resident Engineer (THD)
SILVUS, Clyde F., Austin
Supv. Operations Engineer (THD)
SIMMONS, W. E., Beaumont
District Engineer (THD)
SIMPSON, J. W., Jacksboro
Assoc. Resident Engineer (THD)
SKEWIS, Don E., Corpus Christi
Engineering Assistant (THD)
SMITH, George J., Abilene
Assist. District Engineer (THD)
SMITH, Gilbert, Austin
Texas Railroads
SMITH, Henry T., Anahuac
Chief Inspector (THD)
SMITH, Homer P.R., Jasper
Resident Designer (THD)
SNEDECOR, T. C., Houston
Senior Inspector (THD)
SNELL, Jesse A., Waco
Assist. District Engineer (THD)
SNYDER, J. F., Pharr
District Engineer (THD)
SPARKS, L. O., Lubbock
Portland Cement Assn.
SPENCER, R. Q., Austin
Sr. Maintenance Engineer (THD)
SPILLER, Jamie M., Tyler
Humble Oil & Refining Co.
STAPLES, Jack D., Austin
Designing Engineer (THD)
STAPLETON, G. J., Paris
Engineering Assistant (THD)
STAUBACK, Arnold, Austin
Sr. Field Engineer (THD)
STEWART, Lloyd W., Junction
Senior Inspector (THD)
ST. JOHN, R. H., Houston
Designing Engineer (THD)
STOCKTON, H. P., Jr., Austin
Mgr. Highway Planning Survey (THD)
STOCKTON, Wm. L., Lufkin
Resident Engineer (THD)
STONE, J. R., Stephenville
Engineering Assistant (THD)
STORK, Oliver F., Houston
Resident Engineer (THD)
STROUD, Clark C., Sonora
Sr. Resident Engineer (THD)
STUBBLEFIELD, D. L., Austin
Bureau of Public Roads
STUTZ, Fred, Austin
Assoc. Design. Engineer (THD)

SUTER, W. E., Jasper
Sr. Resident Engineer (THD)
SWAIN, Mark S., Port Arthur
Sr. Resident Engineer (THD)
SWATZELL, Jack D., Cleburne
Allhands & Swatzell, Inc.
SWINDLE, Joe M., Sherman
Engineering Assistant (THD)
SYPHRETT, D. W., Anahuac
Sr. Resident Engineer (THD)
TAYLOR, Auburn G., Fort Worth
Assoc. Resident Engineer (THD)
TAYLOR, H. G., Houston
Chief Inspector (THD)
TAYLOR, J. R., Jr., McKinney
Sr. Resident Engineer (THD)
TAYLOR, Tom H., Austin
Supt., Information Services (THD)
TEAMER, John A., Navasota
Assoc. Resident Engineer (THD)
TERRY, Russell B., Orange
Resident Engineer (THD)
THAMES, C. B., Hearne
Sr. Resident Engineer (THD)
THOMPSON, W. A., Nacogdoches
Nacogdoches County
THORBURROW, R.S., Denison
Assoc. Resident Engineer (THD)
TOWLES, Henry C., Dallas
Uvalde Construction Co.
TRAYLOR, Ed L., Conroe
Chief Inspector (THD)
TRENT, Cecil M., Houston
Senior Draftsman (THD)
TUCKER, R. W., Lubbock
Asst. District Engineer (THD)
TURNER, W. W., Austin
Sr. Operations Supervisor (THD)
TURNER, R. V., Voth
Resident Inspector (THD)
VANCE, Arch A., Bryan
Resident Engineer (THD)
VAN LONDON, W. J., Houston
Engineer-Manager (THD)
VAUGHN, W. A., Mt. Pleasant
American Liberty Oil Co.
VOLZ, Arthur C., Laredo
Resident Engineer (THD)
VOURCOS, M. E., Brownsville
Engineering Assistant (THD)
WACHTSTETTER, C.E., Houston
Engineering Assistant (THD)
WAGENER, Jean S., McAllen
Office Engineer (THD)
WALKER, Arthur V., Austin
Bureau of Public Roads
WALKER, John P., Austin
Area Supervisor (THD)
WALL, Jr., G.C., Mineral Wells
Resident Designer (THD)
WALLACE, Hugh A., Dallas
Asphalt Institute
WALLER, John A., Austin
Chief Engr. of Const. & Maint. (THD)
WALSMITH, F. W., Corpus Christi
Assoc. Resident Engineer (THD)

WARD, Delbert R., San Antonio
H. B. Zachry Co.
WARD, Robert E., Monahans
Sr. Maintenance Foreman (THD)
WARD, W. V., Houston
Chief Inspector (THD)
WATSON, Verne, Fort Worth
Watson Foundation
WEBB, T. H., Austin
Ret. Asst. State Highway Engr. (THD)
WEISINGER, B.W., Gilmer
Sr. Party Chief (THD)
WELBORN, M. C., Austin
Ret. District Engineer (THD)
WEMPLE, Fred A., Midland
Member, State Highway Comm.
WHARTON, L.M., Hondo
Senior Inspector (THD)
WHITE, Jim L., Houston
Portland Cement Assn.
WHITMAN, W. C., Dallas
Sr. Resident Engineer (THD)
WHITSON, J. W., Ft. Stockton
Resident Engineer (THD)
WILCOXSON, A. G., Kaufman
Sr. Maintenance Foreman (THD)
WILKES, W. Jack, Austin
Bureau of Public Roads
WILEY, George L., Austin
Welding Specialist (THD)
WILLIAMSON, D.D., Austin
American Liberty Oil Co.
WILLIAMSON, N. E., Austin
Area Supervisor (THD)
WILLIS, Leo K., San Angelo
Asst. District Engineer (THD)
WILLSON, J. D., Lubbock
Sr. Maintenance Superintendent (THD)
WILSON, Fred B., Conroe
Party Chief (THD)
WILSON, Thomas J., Austin
Assoc. Design. Engineer (THD)
WOOD, Burton R., Floresville
Sr. Maintenance Foreman (THD)
WOOD, Clifford, El Paso
Sr. Resident Engineer (THD)
WOOD, Laurence E., El Paso
Sr. Resident Engineer (THD)
WOOD, Thomas K., Austin
Designing Engineer (THD)
WORD, Claud H., Hillsboro
Sr. Resident Engineer (THD)
WOOLDRIDGE, G. D., Wellington
Senior Instrumentman (THD)
WORRELL, M. E., Junction
WRIGHT, Joe E., Austin
Asst. Traffic Manager (THD)
WRIGHT, S. R., College Station
Civil Engr. Dept., A & M
YEOMAN, Ray C., Lufkin
Texas Foundries, Inc.
YORK, J. M., Lufkin
Asst. District Engineer (THD)
YOUNG, E. R., Beaumont
Supv. Urban Engineer (THD)
YOUNGBLOOD, W. L., Pecos
Sr. Maintenance Foreman (THD)
YOUNGS, G. A., Atlanta
Asst. District Engineer (THD)

IN MEMORIAM

RESOLUTION

WHEREAS, since our last assembled meeting, Almighty God in His infinite wisdom, has called from this life the following associates:

G. RAYMOND ABNEY
R. S. FRAZELL, JR.
WESLEY HALL
CHAS. S. HENNING, JR.
C. P. HUNTER
J. G. IRBY
ARTHUR KNICKERBOCKER
CHARLES H. NEWELL
R. C. PHILLIPS
FELIX REEVES
DEWEY SPENCER
WAYNE D. TINER

WHEREAS, these men throughout the years by their work and character earned the respect and friendship of the members of this group, and contributed much to the advancement of highway development in this State, and

WHEREAS, it is the mutual desire to honor their memories;

NOW THEREFORE, BE IT RESOLVED by the membership of the Twenty-Sixth Annual Highway Short Course assembled at the Agricultural and Mechanical College of Texas, on March 6, 1952, that this expression of our sincere appreciation of their accomplishments and character, and as unanimously adopted in this formal session, be placed in the official records of the Twenty-Sixth Annual Highway Short Course and that the families of these men be furnished a copy of this resolution.

